DEPARTMENT OF TRANSPORTATION

Anchorage of Epoxy-Coated Rebar Using Chemical Adhesives

Benjamin Z. Dymond, Principal Investigator
Department of Civil Engineering
University of Minnesota Duluth

February 2019

Research Project
Final Report 2019-07

LRRB
LOCAL
ROAD RESEARCH
BOARD

To request this document in an alternative format, such as braille or large print, call 651-366-4718 or 1-800-657-3774 (Greater Minnesota) or email your request to ADArequest.dot@state.mn.us. Please request at least one week in advance.

Technical Report Documentation Page

1. Report No. 2019-07	2.	3. Recipients Accession No.	
4. Title and Subtitle Anchorage of Epoxy-Coated Rebar Using Chemical Adhesives		5. Report Date February 2019 6.	
7. Author(s) Connor Mills and Benjamin Z. Dymond		8. Performing Organization Report No.	
9. Performing Organization Name and Address Department of Civil Engineering University of Minnesota Duluth 1405 University Drive Duluth, MN 55812		10. Project/Task/Work Unit No. CTS #2018032 11. Contract (C) or Grant (G) No. (C) 1003325 (wo) 54	
12. Sponsoring Organization Name and Address Local Road Research Board Minnesota Department of Transportation Research Services & Library 395 John Ireland Boulevard, MS 330 St. Paul, Minnesota 55155-1899		13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code	
15. Supplementary Notes http:// mndot.gov/research/reports/2019/201907.pdf			

16. Abstract (Limit: 250 words)

Post-installed reinforcement is used to connect a new concrete member to an existing concrete structure. Typically, uncoated rebar post-installed with a chemical adhesive is used in these applications, which may lead to corrosion. Departments of Transportation and local bridge owners have used and continue to use epoxy-coated rebar in post-installed applications due to its inherent corrosion resistance. Unfortunately, chemical adhesive manufacturers provide tensile strengths of their products for use with uncoated rebar and not epoxy-coated rebar. This work examined what effects the epoxy coating had on the tensile pullout strength and compared the results for epoxy-coated and uncoated rebar. Two slabs were constructed. One slab contained epoxy-coated rebar post-installed using four different chemical adhesive products and the other slab contained uncoated rebar post-installed using the same four different chemical adhesive products. Results indicated that the epoxy coating slightly reduced the tensile pullout strength of the post-installed rebar. The ratio of the tensile pullout strength of the epoxy-coated reinforcing bars to the tensile pullout strength of the uncoated reinforcing bars ranged from 0.94 to 1.05 and varied based on the chemical adhesive manufacturer. Results from *t*-test analyses indicated that differences in the tensile pullout strength for epoxy-coated rebar compared to uncoated rebar were statistically different when using three of the four chemical adhesives during installation. Recommendations were made to include a modification factor when calculating bond strength for an epoxy-coated reinforcing bar post-installed using chemical adhesives and to raise the MnDOT-specified uncracked bond stress (τ_{uncr}) of 1,000 psi or use the manufacturer published values for τ_{uncr}.

17. Document Analysis/Descriptors Reinforcing bars, Adhesives, Epoxy Coating, T test, Tensile Strength, Pull out Test, Tensile Tests		18. Availability Statement No restrictions. Document available from: National Technical Information Services, Alexandria, Virginia 22312	
19. Security Class (this report) Unclassified	20. Security Class (this page) Unclassified	21. No. of Pages 133	22. Price

Anchorage of Epoxy-Coated Rebar Using Chemical Adhesives

FINAL REPORT

Prepared by:

Connor Mills
Benjamin Z. Dymond
Department of Civil Engineering
University of Minnesota Duluth

February 2019

Published by:

Minnesota Department of Transportation
Research Services & Library
395 John Ireland Boulevard, MS 330
St. Paul, Minnesota 55155-1899

ACKNOWLEDGMENTS

Materials were generously provided for the project by the following groups: ABC Coating Co., Adhesive Technology Corporation, Hilti Inc., ITW Red Head, and Powers Fasteners. Equipment, labor, and expertise were generously provided by PCiRoads, LLC.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Post-installed reinforcement is used to connect a new concrete member to an existing concrete structure. Typically, uncoated rebar post-installed with a chemical adhesive is used in these applications, which may lead to corrosion. Departments of transportation and local bridge owners have used and continue to use epoxy-coated rebar in post-installed applications due to its inherent corrosion resistance. Unfortunately, chemical adhesive manufacturers provide tensile strengths of their products for use with uncoated rebar and not epoxy-coated rebar.

The primary objective of this research project was to determine the effect of the epoxy-coating on the tensile pullout strength of reinforcing bars post-installed with a chemical adhesive. Furthermore, a secondary objective was to investigate and clarify the Minnesota Department of Transportation (MnDOT) design procedure for reinforcing bars post-installed with a chemical adhesive in two example applications. To achieve the project objectives, a laboratory study was conducted, which involved casting and testing representative sample specimens that included both epoxy-coated reinforcing bars and traditional uncoated reinforcing bars. Two slabs were constructed. One slab contained epoxy-coated rebar post-installed using four different chemical adhesive products and the other slab contained uncoated rebar post-installed using the same four different chemical adhesive products. A total of 48 tests were conducted to investigate the effect of the specified concrete strength (4,000 psi), reinforcing bar size (#5), embedment depth (5 in. or $8d_b$), reinforcing bar type (epoxy-coated and uncoated), and type of chemical adhesive. The procedure for the tensile pullout strength test of the post-installed reinforcing bars followed guidance from ACI 355.4 (2011) and ASTM E488 (2015).

Results are presented showing the differences between the ultimate tensile pullout strength of epoxy-coated and uncoated reinforcing bars; the difference between the ultimate tensile pullout strength of epoxy-coated reinforcing bars and the manufacturer published bond strength with uncoated reinforcing bars; and the difference between the ultimate tensile pullout strength of epoxy-coated reinforcing bars and the MnDOT-specified bond strength with uncoated reinforcing bars. Results indicated that the epoxy coating slightly reduced the tensile pullout strength of the post-installed rebar. The ratio of the tensile pullout strength of the epoxy-coated reinforcing bars to the tensile pullout strength of the uncoated reinforcing bars ranged from 0.94 to 1.05 and varied based on the chemical adhesive manufacturer. Results from *t*-test analyses indicated that differences in the tensile pullout strength for epoxy-coated rebar compared to uncoated rebar were statistically different when using three of the four chemical adhesives during installation.

To develop recommendations, the standard deviation of each average ultimate tensile pullout strength was added or subtracted to the average ultimate tensile pullout strength of both the epoxy-coated or uncoated reinforcing bars to increase the ratio between the two reinforcing bar types. This methodology created more extreme ratios of the ultimate tensile pullout strength of the epoxy-coated reinforcing bars to the ultimate tensile pullout strength of the uncoated reinforcing bars. The ratios ranged from 0.88 to 0.99. Furthermore, to encompass 99% of the normal distribution, the average ultimate tensile pullout strengths of both the epoxy-coated and uncoated reinforcing bars were reduced by three standard deviations as another means to investigate the ratio of average ultimate tensile pullout strength between the two types of bars, and the minimum ratio was 0.89.

Two recommendations were made. The first recommendation was to include a modification factor when calculating bond strength for an epoxy-coated reinforcing bar post-installed using chemical adhesives ($\psi_{e,Na}$). Application of this modification factor would be similar to the method used in the development length equation from ACI 318 (2014) for cast-in-place reinforcement. For epoxy-coated reinforcing bars post-installed using chemical adhesives, it was recommended that $\psi_{e,Na}$ = 0.9 and be applied to ACI 318-14 Equations 17.4.5.1a and 17.4.5.1b. The bar coating modification factor value of 0.9 was chosen because it encapsulates the lowest ratio of the ultimate tensile pullout strength of the epoxy-coated reinforcing bars to the ultimate tensile pullout strength of the uncoated reinforcing bars from the laboratory experimental program, while still providing a built-in factor of safety. The value of 0.9 also nearly bounded the 0.88 and 0.89 ratios from the other two methods of analysis using the standard deviation.

The second recommendation was that MnDOT raise the specified uncracked bond stress (τ_{uncr}) of 1,000 psi or use the manufacturer published values for τ_{uncr}. The back-calculated τ_{uncr} based on the minimum experimental epoxy-coated ultimate tensile pullout strength was approximately 283% more than the current τ_{uncr} specified by MnDOT. If MnDOT were to use the manufacturer published values for τ_{uncr}, its designs would better follow the requirements of Section 5.13.2.3 from the AASHTO LRFD Bridge Design Specifications (2017) and Section 17.4.5.2 from ACI 318 (2014).

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

The widespread use of concrete as a building material has generated the need to attach new members or items to an existing concrete structure. This is typically done through the use of concrete anchors. According to ACI 318 (2014), concrete anchors are defined as a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads to the concrete. Uses of concrete anchors can range from attaching bike racks to concrete walls to attaching existing structural concrete walls to new structural concrete walls.

Concrete anchors are divided into two groups based on installation timing: cast-in-place anchors and post-installed anchors. Cast-in-place anchors are installed before the concrete is hardened and post-installed anchors are installed into existing, hardened concrete. Post-installed concrete anchors are divided into two groups based on the method of restraining the post-installed anchor: bonded and mechanical. Bonded post-installed concrete anchors are divided into two groups by bonding agent: chemical adhesive and grouted (Zamora et al., 2003). The different groups of anchors are shown in Figure 1.1. Anchorage post-installed with a chemical adhesive can be comprised of different anchor elements (e.g., threaded rod, internally threaded sleeve, or reinforcing bar). When the anchor element is comprised of reinforcing bars, the system can be referred to as reinforcing bars post-installed with a chemical adhesive. The purpose of this research was to specifically investigate reinforcing bars post-installed with a chemical adhesive, which have become popular because of their flexibility in retrofit construction applications and in new construction applications. Reinforcing bars post-installed with a chemical adhesive are installed by drilling holes into hardened concrete, using a specific process to clean the hole, injecting a chemical adhesive into the cleaned hole, and inserting a reinforcing bar into the hole (Cook et al., 1998).

Typically, uncoated black reinforcing bar is used when reinforcing bars are post-installed with a chemical adhesive. However, use of uncoated reinforcing bars in bridge applications may lead to corrosion. Frequently, bridge owners specify that bridge components require some sort of corrosion protection to ensure long-term durability (Dickey, 2011). For the past 30 years, epoxy-coated reinforcing bars have been specified by most state departments of transportation (DOT) for components used in bridge applications because of their durability and competitive life-cycle cost (Hartt et al., 2007). The epoxy-coating forms a barrier between the steel and any corrosive environment associated with service conditions, such as road salts, acids, or carbonation. Utilization of epoxy-coated reinforcing bars has improved the corrosion resistance of reinforcing bars. The usage of epoxy-coated reinforcing bars has increased, and they are now being used for more applications in reinforced concrete construction (Choi et al., 1990). For example, epoxy-coated reinforcing bars are being post-installed with a chemical adhesive in traffic barrier and pier crash strut retrofits (Ehrlich, 2017). According to Dickey (2011), epoxy-coated reinforcing bars post-installed with a chemical adhesive are used by state DOT’s, but they may have different tensile pullout strengths compared to uncoated reinforcing bars post-installed with a chemical adhesive. Due to this issue, chemical adhesive manufacturers have stopped providing a warranty on their products when they are used with epoxy-coated reinforcing bars (Ehrlich, 2017).

1.2 RESEARCH OBJECTIVES

The primary objective of this research project was to determine the effect of the epoxy-coating on the tensile pullout strength of reinforcing bars post-installed with a chemical adhesive. Furthermore, a secondary objective was to investigate and clarify the Minnesota Department of Transportation (MnDOT) design procedure for reinforcing bars post-installed with a chemical adhesive in two example applications. To achieve the project objectives, a laboratory study was conducted, which involved casting and testing representative sample specimens that included both epoxy-coated reinforcing bars and traditional uncoated reinforcing bars. The results from the pullout testing of epoxy-coated reinforcing bars were directly compared to both the results from pullout testing of uncoated reinforcing bars and the manufacturer published tensile strength values for traditional uncoated reinforcing bars. To obtain direct comparisons, both types of reinforcing bars were post-installed using the same chemical adhesives. The final outcome was to determine if epoxy-coated reinforcing bars could be used in post-installed bridge barriers, post-installed bridge pier crash struts, or other applications where reinforcing bars need to be post-installed.

1.3 REPORT ORGANIZATION

Chapter two presents a literature review that examines code and specification provisions related to epoxy-coated reinforcing bars post-installed with a chemical adhesive, test results on epoxy-coated reinforcing bars post-installed with a chemical adhesive, test results of cast-in-place epoxy-coated reinforcing bars, and results from a survey of U.S. state DOT's used to determine where epoxy-coated reinforcing bars are used in post-installed applications. Chapter three discusses the experimental program, which includes a description of the test setup and testing procedure. Chapter four presents the results and discussion of the laboratory experimental program. In chapter five, conclusions are drawn from the experimental program, the discussion is augmented with discrepancies from the literature and results, and final recommendations are made. Appendix A presents a design flowchart and calculations for two applications (post-installed bridge barrier and post-installed bridge pier crash strut) where uncoated reinforcing bars post-installed with a chemical adhesive are used. Appendix B presents plots of the applied tensile load vs. displacement from experimental testing.

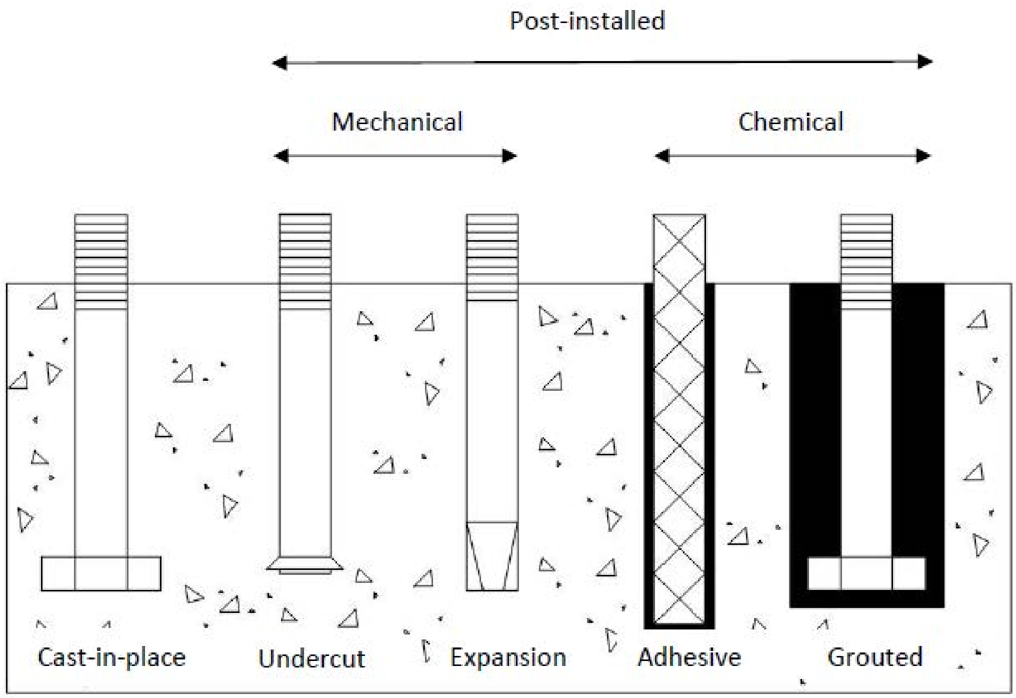


Figure 1.1 Groups of anchors

CHAPTER 2: LITERATURE REVIEW

2.1 CODES AND SPECIFICATIONS

To understand the topic of epoxy-coated reinforcing bars post-installed with a chemical adhesive, it is important to know how the current codes and specifications address the topic. Because using epoxy-coated rebar as post-installed reinforcement in concrete is relatively new, many of the current codes and specifications do not address it. The codes and specifications may address post-installed uncoated reinforcing bars, or more commonly, just address post-installed anchors.

2.1.1 Model Code for Concrete Structures (2010)

The European Committee for Concrete (CEB) and The International Federation for Prestressing (FIP) specify the design of post-installed reinforcement in the Model Code for Concrete Structures (CEB and FIP, 2010). The Model Code for Concrete Structures notes that post-installed reinforcing bar connections are permissible if they follow the design provisions for cast-in-place reinforcing bars. The systems used to post-install reinforcing bars under the Model Code for Concrete Structures (e.g., reinforcing bar, adhesive, hole cleaning tool, and printed manufacturer instructions) must be approved through an independent approval process. Post-installed reinforcing bar connections designed using the Model Code for Concrete Structures must consider the following: inspection of the drilled holes, larger minimum concrete cover compared to cast-in-place reinforcing bars, larger minimum clear spacing compared to cast-in-place reinforcing bars, limited compressive strength, and special requirements for fire safety.

2.1.2 Canadian Highway Bridge Design Code (2014)

Section 8.16.7 of the Canadian Highway Bridge Design Code S6 (2014) entitled "Anchorage of Attachments" broadly discusses cast-in-place, grouted, and adhesive anchorage. However, the code does not specifically address tensile load being transferred to post-installed anchorage via a chemical adhesive, and it does not provide a method to calculate the bond strength of a reinforcing bar post-installed with a chemical adhesive. The code outlines how anchors transfer tensile loads from the anchor to the concrete by one of the following methods: an anchor head at the base of the anchor, a deformed reinforcing bar cast-in-place with a hook, a straight deformed cast-in-place reinforcing bar, or a method approved by the Canadian Highway Bridge Design Code. The Canadian Highway Bridge Design Code does not address how an epoxy coating on deformed reinforcing bars affects the bond strength or anchorage.

2.1.3 AASHTO LRFD Bridge Design Specifications (2017)

The American Association of State Highway and Transportation Officials (AASHTO) addresses anchors in Section 5.13 of the 8th Edition of the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (2017). AASHTO specifies that anchors shall be designed, detailed, and installed according to the provisions of the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete, ACI 318 (2104). The provisions of ACI 318-14 are discussed in detail in Section 2.1.4. ACI 318-14 excludes impact loads from the scope of the post-installed anchor provisions. However, AASHTO allows use of ACI 318-14 provisions for impact load situations.

AASHTO notes that corrosion control shall be considered for any anchor application exposed to environmental elements. Typical corrosion control measures include the use of coatings or corrosion resistant materials (e.g., epoxy-coated reinforcing bars). AASHTO specifies that the manufacturer's literature must document that the adhesive used is compatible with the type and extent of any coating used for adhesive anchors (e.g., epoxy-coated reinforcing bars post-installed with a chemical adhesive).

2.1.4 ACI 318 Building Code Requirements for Structural Concrete (2014)

ACI first added post-installed chemical adhesive anchorage design into the code requirements with ACI 318 Appendix D (2011). When ACI 318 (2014) was published, the post-installed anchorage design requirements from Appendix D were moved into Chapter 17 of the code and updated. Anderson and Meinheit (2014) noted that the reasons for minimally updating the Appendix D material were twofold. The first reason was because the anchor design provisions were only three years old, and the second reason was a desire to keep the code similar for design professionals.

Section 17.4.5 of ACI 318-14 entitled "Bond Strength of Adhesive Anchor in Tension" specifies how to calculate the bond strength of a chemical adhesive anchor acting in tension. ACI 318-14 presents two equations: one for a single chemical adhesive anchor and another for a group of chemical adhesive anchors. According to ACI, two separate equations were included because chemical adhesive anchors that exhibit bond failures when loaded individually may exhibit concrete failures when loaded in a group. The scope of this research was to examine a single anchor in tension, so group effects were not discussed. Figure 2.1 displays the geometry needed to calculate the nominal bond strength in tension of a single chemical adhesive anchor. The equation used to calculate the bond strength of a single anchor in tension is given in ACI 318-14 Section 17.4.5.1 and has the form:

$$N_a = \frac{A_{Na}}{A_{Nao}} \psi_{ed,Na} \psi_{cp,Na} N_{ba} \qquad 2.1$$

Where:

N_a = The nominal bond strength in tension of a single chemical adhesive anchor (lb)

A_{Na} = The projected influence area of a single chemical adhesive anchor that shall be approximated as a rectilinear area that projects outward a distance c_{Na} from the centerline of the chemical adhesive anchor (in.^{2})

A_{Nao} = The projected influence area of a single chemical adhesive anchor with an edge distance equal to or greater than c_{Na} (in.^{2})

c_{Na} = The projected distance from the center of an anchor shaft on one side of an anchor required to develop the full bond strength of a single chemical adhesive anchor (in.)

$\psi_{ed,Na}$ = Modification factor for edge effects

$\psi_{cp,Na}$ = Modification factor for use in uncracked concrete without supplementary reinforcement to account for splitting

N_{ba} = The basic bond strength of a single chemical adhesive anchor (lb)

To use Equation 2.1, the values of c_{Na}, A_{Nao}, $\psi_{ed,Na}$, $\psi_{cp,Na}$, and N_{ba} need to be calculated using the following equations, which are given in ACI 318-14:

$$c_{Na} = 10d_a\sqrt{\frac{\tau_{uncr}}{1100}} \quad (2.2)$$

$$A_{Nao} = (2c_{Na})^2 \quad (2.3)$$

$$\begin{gathered}\text{If } c_{a,min} \geq c_{Na} \text{ then, } \psi_{ed,Na} = 1.0 \\ \text{If } c_{a,min} < c_{Na} \text{ then, } \psi_{ed,Na} = 0.7 + \frac{c_{a,min}}{c_{Na}}\end{gathered} \quad (2.4)$$

$$\begin{gathered}\text{If } c_{a,min} \geq c_{ac} \text{ then, } \psi_{cp,Na} = 1.0 \\ \text{If } c_{a,min} < c_{ac} \text{ then, } \psi_{cp,Na} = \frac{c_{a,min}}{c_{ac}} \\ \psi_{cp,Na} \geq \frac{c_{Na}}{c_{ac}}\end{gathered} \quad (2.5)$$

$$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef} \quad (2.6)$$

Where:

d_a = Diameter of anchor (in.)

τ_{uncr} = Characteristic bond stress in uncracked concrete (psi)

$c_{a,min}$ = Minimum distance from center of anchor to edge of concrete (in.)

c_{ac} = Critical edge distance required for post-installed anchors in uncracked concrete without supplementary reinforcement. For reinforcing bars post-installed with a chemical adhesive, c_{ac} = 2 h_{ef} (in.).

h_{ef} = Effective embedment length of anchor (in.)

λ_a = Modification factor for lightweight concrete

τ_{cr} = Characteristic bond stress in cracked concrete (psi)

The basic bond strength of a single chemical adhesive anchor (N_{ba}) is the bond strength before any modification factors are applied. The modification factor for use in uncracked concrete without supplementary reinforcement ($\psi_{cp,Na}$) is required because concrete splitting can occur if supplementary reinforcement is not provided, which reduces the bond strength. An example of supplementary reinforcement is provided in Figure 2.2. The modification factor for lightweight concrete (λ_a) is 0.45 or 1.0 for lightweight or normal weight concrete, respectively. The effective embedment length of an anchor is how deep the anchor (h_{ef}) is installed into the concrete and is specified by the design engineer.

Section R17.4.5.2 of ACI 318-14 notes that the bond strength of chemical adhesive anchors follows a uniform bond stress model, which is based on results from Cook et al. (1998). Cook et al. examined various design models for adhesive anchors and ultimately recommended a uniform bond stress model because it provided an accurate fit to test reports documenting the behavior of adhesive anchors in European, United States, and Japanese literature. Section R17.4.5.2 of ACI notes that the bond strength is valid for bond failures that occur between the concrete and the adhesive, as well as between the anchor and the adhesive. The values for τ_{uncr} and τ_{cr} are published by the chemical adhesive manufacturer in accordance with ACI 355.4 (2011). ACI 318-14 permits use of τ_{uncr} instead of τ_{cr} in Equation 2.6 when analysis indicates that there will be no cracking at service loads. The bond strength values published by the adhesive manufacturer (τ_{uncr} and τ_{cr}) are applicable for use with threaded rod or uncoated reinforcing bars, but not with epoxy-coated reinforcing bars. ACI 318-14 does not address how the epoxy-coating affects the bond between the post-installed reinforcing bar and the chemical adhesive.

When adhesive manufacturer published values for τ_{uncr} and τ_{cr} are unknown, ACI 318-14 provides lower-bound values, which are given in Table 2.1. There are assumptions inherent in the use of the lower-bound values, including: duration that the load is applied to the anchor, whether the concrete is cracked, size of the anchor, drilling method used to install the anchor, degree of concrete saturation due to water at the time of installation, concrete temperature at the time of anchor installation, concrete age at the time of anchor installation, peak concrete temperature during service life of the anchor, and chemical exposure of the anchor during its service life. The lower-bound values for τ_{uncr} and τ_{cr} do not make any assumptions about anchor coating, so it is unclear if they can be used with epoxy-coated reinforcing bars.

Instead of using the lower-bound values from Table 2.1 or the manufacturer published values, MnDOT policy specifies values for τ_{uncr} and τ_{cr} (MnDOT, 2016). The MnDOT-specified values for τ_{uncr} and τ_{cr} are 1,000 psi and 500 psi, respectively. These values are incorporated into two sample calculations of reinforcing bars post-installed with a chemical adhesive in Appendix A. The first sample calculation is a post-installed bridge barrier and the second is a post-installed bridge pier crash. The example calculations followed the methods outlined in the AASHTO LRFD Bridge Design Specifications (2017), ACI 318 (2014), and MnDOT LRFD Bridge Design Manual (2017). The sample calculations are preceded by a flowchart to provide an engineer with guidance about the design of reinforcing bars post-installed using chemical adhesives.

2.1.5 A23.3 Design of Concrete Structures (2014)

The Canadian Standards Association (CSA) addresses post-installed chemical adhesive anchorage design in A23.3 (2014). A23.3 presents the same equations as ACI 318 (2014) to calculate bond strength of a post-installed chemical adhesive anchor in tension. Similar to ACI 318-14, A23.3 requires chemical adhesive anchors to be qualified for use by ACI 355.4 (2011) and the values for τ_{uncr} and τ_{cr} to be reported in accordance with ACI 355.4. The only difference between A23.3-14 and ACI 318-14 is the fact that A23.3 uses an additional resistance modification factor (R) and a material resistance factor for concrete (ϕ_c) in Equation 2.6, whereas ACI 318-14 only applies a strength reduction factor (ϕ) to

Equation 2.6. For example, the factored resistance must be greater than demand in ACI 318-14 ($\phi N_n \geq N_u$), whereas the equation takes the form of ($\phi c R_{Nn} \geq N_u$) in A23.3-14. The magnitude of bond strength reduction is the same in both methods despite the different processes used by ACI 318-14 and A23.3-14 to apply resistance factors.

2.1.6 ACI 355.4 Qualification of Post-Installed Adhesive Anchors in Concrete (2011)

When ACI 318 (2011) introduced the design methodology for uncoated reinforcing bars post-installed with a chemical adhesive into the building code requirements, the Institute also published the Qualification of Post-Installed Adhesive Anchors in Concrete (ACI 355.4-11), which is the standard that qualifies post-installed chemical adhesive anchor systems for use in practice. ACI 355.4-11 defines chemical adhesive anchor systems as consisting of the following components:

- Anchor (e.g., threaded rod, internally threaded sleeve, or reinforcing bar)
- Proprietary chemical adhesive compounds in combination with a mixing and delivery system
- Accessories for cleaning the drilled hole
- Manufacturer printed instructions for adhesive anchor installation

ACI 355.4-11 prescribes testing requirements for post-installed chemical adhesive anchors intended for use in concrete under the design provisions of ACI 318 (2014). The assessment and testing criteria in ACI 355.4-11 are for various uses, which include sustained loading, seismic loading, aggressive environments, reduced or elevated temperatures, usage in only uncracked concrete, or usage in cracked and uncracked concrete. Along with the assessment and testing criteria for various uses, ACI 355.4-11 provides testing criteria for establishing the characteristic bond strength (τ_{uncr} and τ_{cr}), reductions in capacity for adverse conditions, and jobsite quality control requirements. Anchor system evaluation is based on four types of tests:

1. Identification tests to evaluate anchor compliance with manufacturer specifications
2. Reference tests to obtain baseline values for the evaluation of reliability and service-condition test results
3. Reliability tests to assess anchor sensitivity to adverse installation conditions and long-term loading
4. Service-condition tests to establish anchor performance under expected service conditions

Different anchor types (e.g., threaded rod, internally threaded sleeve, or reinforcing bar) and chemical adhesive types (e.g., vinylester, epoxy, and hybrid systems) may exhibit a range of performance characteristics. The strength capacity of adhesive anchors is sensitive to variations in installation and service-condition parameters (e.g., hole cleaning, installation orientation, and cracked concrete characteristics). Adhesive manufacturers report values for the strength reduction factor (ϕ) in accordance with ACI 355.4-11 Section 17.3.3 and ACI 318-14, which are given in Table 2.2. Strength reduction factors are separated by category and condition. Categories one through three describe differences in the installation sensitivity and reliability of an anchor. The difference between Condition A and Condition B is whether supplementary reinforcement is provided. ACI notes that when supplementary reinforcement is provided, failure will occur in a ductile instead of a brittle manor, thus allowing a higher strength reduction factor.

The strength reduction factors do not address how different chemical adhesive anchor systems may exhibit a range of performance. Variation in the performance of different chemical adhesive anchor systems is addressed in Tables 3.1 through 3.3 of ACI 355.4-11. These tables provide a test program for evaluation of adhesive anchor systems considering the following:

- Presence of water during anchor installation
- Drilling method (e.g., hammer drill or core drill)
- Installation direction
- Installation temperature
- Embedment depth and anchor diameter
- Anchor element type (includes different steel material types such as carbon and stainless steel, different tensile strengths of steel, and different anchor elements such as threaded rod, internally threaded sleeve, and reinforcing bar)
- Environmental use conditions (e.g., dry, wet, and elevated temperatures)
- Chemical exposure (e.g., high alkalinity or sulfur dioxide)
- Concrete condition (e.g., cracked or uncracked)
- Loading (e.g., static, seismic, or sustained)
- Concrete member thickness

A tension test of a single anchor, unconfined, away from any concrete edges, and in uncracked concrete is test 7a in ACI 355.4-11 and is summarized in Table 2.3. Test program 7a is a service-condition test used to determine the characteristic bond stress of a chemical adhesive in uncracked concrete (τ_{uncr}), which is used in ACI 318-14 to calculate the nominal bond strength of a single chemical adhesive anchor in tension. Table 2.3 is a summary of Tables 3.1 through 3.3 from ACI 355.4-11, which describes the testing performed in this research project.

2.1.7 AC 308 Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements (2013)

The Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements (ICC, 2013), which is generally referred as AC 308, was initially adopted in 2006 by the International Code Council (ICC). The purpose of AC 308 is to address the strength design criteria of chemical adhesive anchors compatible with the provisions of ACI 318. The first version of AC 308 (2006) provided design provisions that could be used with ACI 318 and included requirements that accounted for cracked concrete, adverse installation conditions, and sustained tension loads (Hörmann-gast and Silva, 2012). AC 308 now conforms to ACI 355.4 (2011) requirements to avoid having two different standards for acceptance in the anchoring industry (Anderson and Meinheit, 2014). Typically, the revised AC 308 (2013) conforms to ACI 355.4 (2011).

AC 308 is comprised of two parts. The first part is the main body of the criteria, which defines the applications for the criteria, applicable reference standards, and any additional design provisions to be included in the evaluation of chemical adhesive anchors. This part of the criteria conforms to ACI 355.4-11. The second part of AC 308 summarizes amendments, which supersede the applicable portions of ACI 355.4-11. The differences between AC 308 and the content from ACI 355.4-11 include:

- Assessment of chemical adhesive anchor systems for post-installed reinforcing bars
- Assessment of torque-controlled chemical adhesive anchors
- Evaluation tests on anchors to be conducted by an Independent Testing and Evaluation Agency
- Assessment of chemical adhesive anchors for applications that experience wide temperature variations over a short period
- Creating a method to establish the bond stresses as a function of anchor diameter
- Chemical adhesive injection-verification tests
- Clarification on how to conduct confined tests to establish bond stresses

AC 308 discusses the use of reinforcing bars as post-installed chemical adhesive anchor elements in Section 9.3 because ACI 355.4-11 never explicitly gives guidance for the use of reinforcing bars instead of other chemical adhesive anchors. AC 308 notes that the test program from ACI 355.4-11 can be used for evaluation of reinforcing bars as post-installed chemical adhesive anchor elements. AC 308 does not place any requirements on the reinforcing bar to be used, such as steel material types (e.g., carbon or stainless), tensile strength of steel, or different reinforcing bar coatings (e.g., epoxy-coated).

2.1.8 ASTM E488 Standard Test Methods for Strength of Anchors in Concrete Elements (2015)

The American Society for Testing and Materials (ASTM) published the Standard Test Methods for Strength of Anchors in Concrete Elements (ASTM, 2015), which is referred to as E488. ASTM presents test methods to determine the tensile and shear strengths of post-installed and cast-in-place anchors in E488. Both ACI 355.4-11 and AC 308 specify that test members must conform to the requirements of E488, unless noted otherwise. The test methods presented in E488 can be used in cracked or uncracked concrete. The test specimens can be subject to static, seismic, fatigue, or shock loadings. The test specimens can also be subjected to environmental exposure, which include freeze thaw, moisture, decreased temperatures, elevated temperatures, and corrosion.

E488 provides requirements for the testing equipment used to perform tension test 7a from ACI 355.4-11. Figure 2.3 shows an example test setup. These requirements include precision tolerances for the devices used to measure load (1% tolerance of anticipated ultimate load) and displacement (0.001 in.) and requirements on how often the load and displacement should be recorded (once per second). E488 requires that the testing equipment have sufficient capacity to prevent yielding of its components under the anticipated ultimate load, have sufficient stiffness to ensure that the applied tension loads remain parallel to the axis of the anchors, and the tension load plate thickness be at least equal to the diameter of the anchor being tested.

ASTM E488 provides requirements for the concrete test specimen used to evaluate anchors, which include:

- Thickness of at least $1.5h_{ef}$
- Cement type (use Portland cement, if any supplementary cementitious materials or admixtures are used, report them)
- Aggregate type (conform to ASTM C33 Standard Specification for Concrete Aggregates)
- Casting position (horizontally or vertically, if the member is cast vertically, the maximum height shall be 5 ft)

ASTM E488 notes general testing procedures, which include:

- Anchor installation (install the anchors according to the manufacturer's instructions)
- Anchor placement (install anchors in a formed face of the concrete or in concrete with a steel-troweled finish)
- Drill requirements (drill holes with a rotary-percussive hammer drill using carbide-tipped hammer-drill bits)

The testing procedure for load application includes an initial load of up to 5% of the estimated ultimate load to bring the elements into bearing, followed by increasing the load or displacement so failure occurs within three minutes.

2.2 BOND STRENGTH OF ANCHOR SYSTEMS POST-INSTALLED WITH A CHEMICAL

Over the years, there has been research conducted on post-installed anchor systems consisting of reinforcing bars, but these systems did not specifically address the use of epoxy-coated reinforcing bars. Research related to the bond strength of reinforcing bars post-installed with a chemical adhesive does not necessarily apply to the bond strength of epoxy-coated reinforcing bars post-installed with a chemical adhesive, but is important to understand the bond between the rebar, concrete, and chemical adhesive.

2.2.1 Cook, Kunz, Fuchs, and Konz (1998)

In this study, various proposed design models for single adhesive anchor systems were compared and evaluated for accuracy. These anchor systems included reinforcing bars, threaded rods, and internally threaded sleeves. The study examined test reports documenting the behavior of adhesive anchors in European, United States, and Japanese literature; these test reports contained the results from 2,929 tests. The reports consisted of tensile and shear load test data in uncracked and cracked concrete with single anchors and groups of anchors. The test reports distinguished between confined and unconfined tests, tests away from the concrete edges, tests near the concrete edges, and tests on anchor groups. Cook et al. (1998) proposed the following design models:

- Concrete cone model

- Uniform bond stress model
- Bond model neglecting the shallow concrete cone
- Cone models with bond model
- Combined cone/bond model
- Two-interface bond model

The two-interface bond model was based on distinguishing between the adhesive/concrete and adhesive/anchor bond failure modes. This concept was established for uncoated reinforcing bar applications. The test reports that Cook et al. examined from European, United States, and Japanese literature did not adequately distinguish between the adhesive/concrete and adhesive/anchor bond failure modes. According to Cook et al., it was difficult to distinguish between the different failure modes.

Due to the flaws in the two-interface bond model, Cook et al. (1998) eventually concluded that the uniform bond stress model was preferred. Cook et al. stated that the uniform bond stress model is a simplified design model that most accurately indicated the failure load of a given anchor. This is because it provided the best fit to the results in test reports documenting the behavior of adhesive anchors in European, United States, and Japanese literature and agreed with non-linear analytical studies of adhesive anchor systems done by McVay et al. (1993). Cook et al. noted that the uniform bond model is user-friendly and easy to implement. According to Cook et al., implementation of this model would require development of a product acceptance standard that would need to include a series of tension tests to determine the appropriate bond strength of a product and its susceptibility to commonly occurring installation and in-service conditions. The uniform bond stress model was implemented in concrete codes when ACI took the equation presented in Cook et al. (1998) and included it in ACI 318 (2011) to calculate the basic bond strength of an adhesive anchor (Equation 2.6 in this document). ACI 355.4 is the acceptance standard used to qualify post-installed adhesive anchors in concrete that Cook et al. recommended.

2.3 BOND STRENGTH OF EPOXY-COATED REINFORCING BARS POST-INSTALLED WITH A

The design of reinforcing bars post-installed with a chemical adhesive as an anchor per ACI-318 is permitted if the post-installed reinforcing bar system has been qualified by AC 308. The chemical adhesive manufacturer published values for the bond strength of post-installed reinforcing bar systems do not include values for the bond strength of epoxy-coated reinforcing bars. This section discusses the bond strength of epoxy-coated reinforcing bars post-installed with a chemical adhesive.

2.3.1 Meline, Gallaher, and Duane (2006)

Meline et al. (2006) evaluated the performance of epoxy-coated reinforcing bars post-installed with a chemical epoxy adhesive. The study did not directly compare the bond of post-installed epoxy-coated reinforcing bars to the bond of post-installed uncoated reinforcing bars, but the authors compared the bond of post-installed epoxy-coated reinforcing bars to the manufacturers' published design values for uncoated reinforcing bars.

This study performed 90 tests on #6 reinforcing bars in accordance with AC 58 (2010), which was the allowable stress design acceptance criteria for adhesive anchors before the AC 308 (2013). Of the 90 tests performed, 30 of the tests were tensile pullout tests and the rest were seismic tests or creep tests. Of the 30 tensile pullout tests, 10 each were for the three different chemical epoxy adhesives tested: Simpson Strong-Tie SET22, Red Head Epcon Ceramic 6, and Covert Operations CIA-Gel 7000. Of the 10 tensile pullout tests for each of the three different epoxy adhesives, five of the tests were for an embedment depth of 9 in. and five were for an embedment depth of 6.75 in. The tension tests were conducted on an unconfined single reinforcing bar away from concrete edges, rather than on a confined single reinforcing bar away from concrete edges. Because the tension tests were unconfined, concrete breakout failure mode could occur. The test setup was designed in compliance with ASTM E488 (1996).

While performing the tensile pullout tests, the researchers observed that the failures for the epoxy-coated reinforcing bar generally happened when the adhesive debonded from the epoxy coating of the reinforcing bar. During tests with the uncoated reinforcing bars, the adhesive rarely debonded from the reinforcing bar but rather debonded from the concrete. Meline et al. stated that, when compared to the manufacturer published design values, the post-installed epoxy-coated reinforcing bar outperformed the uncoated reinforcing bar for allowable tensile loads for Red Head Epcon Ceramic 6 and Covert Operations CIA-Gel 7000 adhesives. Meline et al. stated that the Simpson Strong-Tie SET22 adhesive underperformed compared to the manufacturer design values, but the results were close enough such that Meline et al. concluded that post-installed epoxy-coated reinforcing bars performed similarly to post-installed uncoated reinforcing bars. The results of this study are listed in Table 2.4, which compares the average ultimate tensile strength of the post-installed epoxy-coated reinforcing bar to the manufacturer published tensile strength of post-installed uncoated reinforcing bars. Table 2.4 also presents the failure modes that were observed during testing, which included: bond failure at the adhesive/reinforcing bar interface, bond failure at the adhesive/concrete interface, concrete breakout, reinforcing bar yielding, and reinforcing bar chuck slip.

There were limitations to this study. It did not evaluate the anchor system in accordance with the current AC 308 (2013) but rather the allowable stress design acceptance criteria AC 58 (2010). This study only looked at epoxy-based chemical adhesives and no other types of adhesives (e.g., vinylester and hybrid systems). Furthermore, the study only compared the results to the manufacturer published test data and did not directly compare experimental data for the bond of post-installed epoxy-coated reinforcing bars to experimental data for the bond of post-installed uncoated reinforcing bars. Despite the limitations, this study provided insight related to how the bond strength may be affected when an epoxy coating was applied to the post-installed reinforcing bar.

2.3.2 Dickey (2011)

A guide for the design of epoxy-coated reinforcing bars post-installed with a chemical adhesive for use in concrete traffic barriers was developed by Dickey (2011) for the Wisconsin Department of

Transportation. A series of impact tests and a static test were conducted on single post-installed reinforcing bars. Tests were conducted on the post-installed reinforcing bars to determine their capacity under an impact load, which caused tension in the post-installed reinforcing bars. Both epoxy-coated and uncoated reinforcing bars were tested to provide a direct comparison of how the coating affected the impact tensile pullout strength. The purpose of the static tension test conducted by Dickey was to determine the relationship of the static tensile pullout strength to the impact tensile pullout strength. This relationship was described by an impact increase factor, which was calculated by dividing the impact tensile pullout strength by the static tensile pullout strength. The impact increase factor in this study was 1.06. Because the impact increase factor was close to 1.0, Dickey concluded that the impact test results from this study were similar to tensile pullout strength results of the post-installed reinforcing bar under static loads.

Dickey performed a series of seven impact tests on single epoxy-coated and uncoated reinforcing bars to determine their tensile pullout strength. The load was applied by a test vehicle that mimicked a car crashing into a replicated traffic barrier at 10 mph. Dickey used jigs that replicated the load transfer mechanisms of traffic barriers to transfer the momentum of the vehicle into impact tensile forces on the post-installed reinforcing bars. All of the tests were conducted on one unreinforced concrete slab with a compressive strength of 6,454 psi. Anchor holes were drilled using a carbide-tipped concrete bit with a rotary hammer drill, and the holes were cleaned by repeated brushing and blowing of compressed air into the hole per the manufacturer's specifications. All of the post-installed epoxy-coated reinforcing bars had an embedment depth of 5.25 in., and the reinforcing bars were ASTM A615 (2016) Grade 60 #5 or #6. The chemical adhesive used was Hilti HIT-RE 500 for the #5 reinforcing bars and Hilti HIT-RE 500-SD for the #6 reinforcing bars; both chemical adhesives were epoxy-based.

After testing, Dickey concluded that epoxy-coatings on the post-installed reinforcing bars decreased the tensile pullout strength by approximately 9%. The average tested impact load ratio for epoxy-coated #5 reinforcing bars to uncoated #5 reinforcing bars was 0.91. The average tested impact load ratio for epoxy-coated #6 reinforcing bars to uncoated #6 reinforcing bars was 0.90. For the #5 reinforcing bars, both the epoxy-coated and uncoated reinforcing bars exhibited a failure mode of steel fracture. For the #6 reinforcing bars, both the epoxy-coated and uncoated reinforcing bars exhibited a failure mode of bond failure between the reinforcing bar and the chemical adhesive. The results of the impact tensile tests on single epoxy-coated and uncoated reinforcing bars are shown in Table 2.5.

2.4 SIMILARITIES BETWEEN REINFORCING BARS POST-INSTALLED WITH A C ADHESIVE AND CAST-IN-PLACE REINFORCING BARS

Research has determined that there are similarities between reinforcing bars post-installed with a chemical adhesive and cast-in-place reinforcing bars. The similarities include the bond between the reinforcement and concrete for cast-in-place reinforcing bars and the bond between the reinforcement, chemical adhesive, and concrete for reinforcing bars post-installed with a chemical adhesive.

2.4.1 Eligehausen, Mahrenholtz, Akguzel, and Pampanin (2012)

Eligehausen et al. (2012) tested post-installed and cast-in-place column-foundation connections. The post-installed column-foundation connection specimens were designed using reinforcing bars post-installed with a chemical adhesive as an anchor system per ACI 318 (2011), and the cast-in-place column-foundation connection was designed using the development length provisions of ACI 318-11. The ratio for the average moment capacity of the column-foundation connection with post-installed reinforcement to the average moment capacity of the column-foundation connection with cast-in-place reinforcement was calculated to be 1.2. Eligehausen et al. concluded that a ratio of 1.2 indicated that the behavior of the cast-in-place specimens and the post-installed specimens were not significantly different. The results of Eligehausen et al. are summarized in Table 2.6.

2.5 EFFECTS OF EPOXY COATINGS ON BOND BETWEEN CAST-IN-PLACE REINFORCING BARS

It is worthwhile to examine the effect an epoxy coating has on the bond between the reinforcement and concrete for cast-in-place reinforcing bars because there is a gap in the literature related to how an epoxy coating affects the bond between reinforcement and adhesives for post-installed reinforcing bars. Furthermore, Eligehausen et al. (2012) concluded that the behavior of the cast-in-place reinforcement and the post-installed reinforcement were similar. ACI 318 (2014) notes that the development length concept is based on the average bond stress over the embedment length of the reinforcement, and this concept has similarities to the uniform bond stress model in ACI 318-14 for post-installed adhesive anchorage. The equation from ACI for development length is given in Equation 2.7. In ACI 318-14, there is a modification factor (ψ_e) that increases the required development length of cast-in-place reinforcing bars with an epoxy coating, as shown in Table 2.7. The epoxy modification factor is based on results from studies by Mathey and Clifton (1976), Johnston and Zia (1982), and Treece and Jirsa (1989). These studies showed that the bond strength was reduced because the epoxy coating on the reinforcing bars prevented adhesion between the reinforcement and the concrete and lowered the coefficient of friction between the reinforcing bar and the concrete.

$$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \tag{2.7}$$

Where:

l_d = Development length (in.)
f_y = Yield strength of reinforcing bars (psi)
ψ_t = Modification factor for casting position
ψ_e = Modification factor for reinforcing bar coating (e.g., epoxy coating)
ψ_s = Modification factor for reinforcing bar size
d_b = Diameter of reinforcing bar (in.)
λ = Modification factor for lightweight concrete

f'_c = Compressive strength of concrete (psi)

c_b = Lesser of the distance from the center of the reinforcing bar to the nearest concrete surface or one-half the center-to-center spacing of the reinforcing bars being developed (in.)

K_{tr} = Transverse reinforcement index (in.)

2.5.1 Mathey and Clifton (1976)

In a study by Mathey and Clifton (1976), the bond strength was determined for a total of 34 cast-in-place specimens: 23 epoxy-coated, six polyvinyl chloride coated, and five uncoated reinforcing bars. The 23 epoxy-coated reinforcing bars had varying coating thickness and different epoxy-coating types. The researchers concluded that when the epoxy-coating thickness was less than 10 mils (0.01 in.), the bond strength of epoxy-coated bars was similar to that of uncoated reinforcing bars. The researchers concluded that when the epoxy-coating thickness was greater than 10 mils, the bond strength of the epoxy-coated reinforcing bars relative to the bond strength uncoated reinforcing bars was reduced enough that it should not be used as a protective coating for reinforcing bars. Currently, ASTM A775 specifies a coating thickness of 7 to 12 mils (0.007 to 0.012 in.) for #5 and smaller reinforcing bars and a coating thickness of 7 to 16 mils (0.007 to 0.016 in.) for #6 and larger reinforcing bars in Standard Specification for Epoxy-Coated Steel Reinforcing Bars (ASTM, 2017). Mathey and Clifton recommended that no modification factor be applied for epoxy-coated reinforcing bars with a coating thickness less than 10 mils, and reinforcing bars with a coating thickness greater than 10 mils should not be used as reinforcement in concrete structures.

2.5.2 Johnston and Zia (1982)

Johnston and Zia (1982) completed a study with the objective of comparing the bond characteristics of cast-in-place epoxy-coated reinforcing bars and cast-in-place uncoated reinforcing bars. Tests were conducted on epoxy-coated reinforcing bars with a coating thickness less than 10 mils (0.01 in.) and uncoated reinforcing bars. These tests were comprised of 40 beam-type flexural bond specimens with #6 and #11 reinforcing bars. Specimens with each size reinforcing bar had three different embedment lengths. The #6 bar had 8, 13, and 18 in. embedment lengths, while the #11 bar had 16, 24, and 30 in. embedment lengths.

After testing, the researchers concluded that the epoxy-coated reinforcing bars had less resistance than the uncoated reinforcing bar. The researchers found the bond strength of the shortest embedment length (8 in.) to be 32% less for epoxy-coated reinforcing bars compared to uncoated reinforcing bars and 15% less for epoxy-coated reinforcing bars compared to uncoated reinforcing bars at the longer development lengths (13, 16, 18, 24, and 30 in.). However, after completing their testing, Johnston and Zia recommended a modification factor of 1.15 be used to increase the development length of epoxy-coated reinforcing bars regardless of the embedment length, which is less than the 1.2 modification factor ACI 318 (2014) specifies for reinforcing bars with sufficient clear cover and clear spacing shown in Table 2.7.

2.5.3 Treece and Jirsa (1989)

Treece and Jirsa (1989) tested 21 beams to compare the bond strength of epoxy-coated reinforcing bars to uncoated reinforcing bars. The reinforcing bars in the beams were lap spliced at midspan, and force was applied to the beams with two point loads at third-points of the span length to achieve a constant midspan moment. The testing program investigated how bond strength was affected by bar size, concrete strength, casting position, and epoxy coating thickness. The testing was completed in nine groups. Within the nine groups, the only variable that changed was whether the reinforcing bar was epoxy-coated or uncoated, and this was done so the bond strength could be compared between the epoxy-coated and uncoated reinforcing bars.

Based on the results of the testing program, the researchers concluded that the bond strength for the epoxy-coated reinforcing bars was 15% less than the bond strength for the uncoated reinforcing bars, which is the same value reported by Johnston and Zia (1982). Similar to Johnston and Zia, Treece and Jirsa also found the reduction in bond strength to be independent of reinforcing bar size. Treece and Jirsa also concluded that the bond strength was independent of the concrete strength. Based on their experimental results, the authors recommended a development length modification factor of 1.15 be used, which is less than the 1.2 modification factor ACI 318 (2014) specifies for reinforcing bars with sufficient clear cover and clear spacing shown in Table 2.7.

Treece and Jirsa hypothesized that the primary reason for the reduction in the bond strength of epoxy-coated reinforcing bars might have been the loss of adhesion between the epoxy coating and the surrounding concrete. The bond strength of uncoated reinforcing bars is higher because they are not subjected to the loss of adhesion.

2.6 USAGE OF EPOXY-COATED REINFORCING BARS POST-INSTALLED WITH A CHEMICAL

A survey was conducted to understand the current usage of epoxy-coated reinforcing bars post-installed with a chemical adhesive by U.S. state DOTs. An online survey was created that contained six questions:

1. Does your agency use post-installed, epoxy-coated rebar attached with a chemical adhesive for anchorage?
2. In what applications do you use post-installed, epoxy-coated rebar attached with a chemical adhesive?
3. What procedure do you use for calculating anchorage bond strength?
    a. ACI 318-14 Chapter 17 (Anchoring to Concrete)
    b. Manufacturer’s literature
        i. (if yes) What manufacturer’s literature does your agency use and do you account for the epoxy coating?
    c. Other process
        i. (if yes) Please explain the other process used by your agency for calculating anchorage bond strength.

4. What type of verification does your agency employ to ensure post-installed anchorage performance? Please describe special requirements for inspection, proof testing, etc.
5. If your agency performs load testing on in-situ adhesive anchors, what is the frequency of testing and magnitude of the test load?
6. Within your agency, who can we contact to gather additional details and information?

The survey was distributed to the AASHTO Committee on Bridges and Structures listserv on November 27, 2017. The responses were gathered over the ensuing two months. After evaluation of the survey responses, follow-up questions were asked when clarification was required. Of the 30 states that responded to the survey, 12 did not use epoxy-coated rebar post-installed with a chemical adhesive, as shown in Table 2.8. The remaining 18 states that used epoxy-coated rebar post-installed with a chemical adhesive are listed in Table 2.9. Furthermore, Table 2.9 shows the applications where epoxy-coated rebar post-installed with a chemical adhesive was used and the procedure used to calculate bond strength. Table 2.10 provides the verification procedures employed to ensure post-installed anchorage performance by 16 of the states that used epoxy-coated rebar post-installed with a chemical adhesive (Anonymous 2 and Anonymous 3 did not specify their verification procedures and could not be reached for follow-up questions).

Five states had a method in place for using epoxy-coated rebar post-installed with a chemical adhesive (California, Iowa, Michigan, Minnesota, and Ohio). California provided guidance for design of uncoated rebar post-installed with a chemical adhesive in their Bridge Design Aid 5-81 (2012). The guidance in the Bridge Design Aid 5-81 included minimum edge distances, minimum embedment depth, drilled hole diameter, shear design strength, and tension design strength. California specified requirements for both uncoated and epoxy-coated rebar post-installed with a chemical adhesive in section 51-1.03E(5) of their Standard Specifications (2015). When the post-installed rebar is epoxy-coated, their Standard Specifications required a 50% increase in the embedment depth specified in the Bridge Design Aid 5-81, and the tensile design strength stayed the same despite the increased embedment depth. The increase in the embedment depth for epoxy-coated rebar post-installed with a chemical adhesive was the result of research performed by the California DOT (Meline and Duane, 2006). However, it was unclear how California decided to increase the embedment depth by exactly 50%. Multiple follow-up phone discussions with the California DOT did not provide additional detail or clarification.

Iowa required that chemical adhesives used to post-install rebar must be approved prior to use with epoxy-coated rebar in accordance with Appendix D of IM 491.11 (2016), which is an internal memorandum governing polymer grouts and adhesive anchors issued on October 18, 2016. Appendix-D of IM 491.11 notes that the approval process shall be based on tension tests from ASTM E488 (2015). Iowa limited the allowable load for all post-installed adhesive anchors to 25% of the ultimate strength. This limitation included epoxy-coated rebar post-installed with a chemical adhesive. A follow-up phone discussion with the Iowa DOT did not yield insight or clarification on why a limit of 25% of the ultimate strength was chosen.

Michigan evaluated the chemical adhesive products used to post-install epoxy-coated rebar for acceptance on their Qualified Products List (2018). This acceptance process starts with a review of the manufacturer's literature and independent test data. The manufacturer's literature was occasionally used for design, but guidance from the Michigan Bridge Design Manual (2017) was used more frequently. Michigan accounted for an epoxy coating by performing quality assurance testing prior to including epoxy-coated rebar on their Qualified Products List. The guidance from the Michigan Bridge Design Manual included installing the rebar to an embedment depth of 12 times the rebar diameter ($12d_b$) and reducing the allowable tension load by applying a safety factor of four to 125% of the rebar yield strength. In other words, the allowable tension load is equal to $(1.25 \cdot f_y \cdot A_s)/4$, where f_y is the yield stress and A_s is the nominal area of the reinforcing steel. A phone conversation with the Michigan DOT clarified that their method for using epoxy-coated rebar post-installed with a chemical adhesive mimics manufacturer’s literature from the 1990’s, which is how long Michigan has had their method in place.

Minnesota used the same method as ACI 318 (2014), but Minnesota deviated from ACI by specifying the uncracked bond strength (τ_{uncr}) to be 1000 psi, the cracked bond strength (τ_{cr}) to be 500 psi, and by using AC 308 (2013) to calculate the critical edge distance, (c_{ac}), within the modification factors for splitting effects ($\psi_{cp,N}$ and $\psi_{cp,Na}$). Finally, Ohio’s method did not require calculation of bond strength for epoxy-coated rebar post-installed with a chemical adhesive because this type of anchorage was only used in non-structural applications.

Of the eight states that used the manufacturer's literature for design, three accounted for epoxy-coated rebar, three did not account for epoxy-coated rebar, one sometimes accounted for epoxy-coated rebar, and one did not specify if epoxy-coated rebar was accounted for (Anonymous 2 could not be reached for follow-up questions). The three states that used the manufacturer's literature and accounted for epoxy-coated rebar were Anonymous 1, Arkansas, and Michigan. Anonymous 1 said they only used epoxy-coated rebar post-installed with a chemical adhesive when it is allowed by the manufacturer. Because Anonymous 1 did not leave any contact information, they could not be reached for follow-up questions. Arkansas noted that they used the manufacturer's design tables for epoxy-coated rebar post-installed with a chemical adhesive, and if no manufacturer design table is available, they used the development length modification factor for epoxy-coated rebar from ACI 318 (2014) to increase the required embedment length. The development length modification factor for epoxy-coated rebar from ACI 318-14 is ψ_e and ranges from 1.0 to 1.5 depending on the type of rebar coating and clear cover/spacing as shown in Table 2.7. Arkansas noted that chemical adhesive products used to post-install epoxy-coated rebar must be approved for use with epoxy-coated rebar and must be on their Qualified Products List (2018).

The three states that used the manufacturer's literature and did not account for the epoxy coating were Kentucky, North Dakota, and Wyoming. The state that used the manufacturer's literature and sometimes accounted for epoxy-coated rebar was Utah. Utah noted that the chemical adhesive and the manufacturer’s literature are submitted on a project specific basis. Utah also noted that the epoxy coating was not always accounted for.

2.7 SUMMARY OF LITERATURE

To understand the topic of epoxy-coated reinforcing bars post-installed with a chemical adhesive, it was important to know how the current codes and specifications address the topic. Because using epoxy-coated reinforcing bars as post-installed reinforcement in concrete is relatively new, many of the current codes and specifications do not address it.

The only current code or specification that specifically addressed the use of epoxy-coated reinforcing bars post-installed with a chemical adhesive was the AASHTO LRFD Bridge Design Specifications (2017). AASHTO specifies that the manufacturer's literature must document that the chemical adhesive used is compatible with the type and extent of coating used on the epoxy-coated reinforcing bars. However, the manufacturer's literature often does not provide this type of required documentation.

The provisions of ACI 318 (2014) do not specifically address epoxy-coated reinforcing bars post-installed with a chemical adhesive, but the code addresses the more general subject of adhesive anchors. ACI 318-14 specifies how adhesive anchors should be designed, detailed, and installed. Other codes and specifications such as AASHTO and CSA A23.3-14 (2014) make modifications to ACI 318-14. ACI 318-14 excludes impact loads from the scope of the post-installed anchor provisions. However, AASHTO allows use of ACI 318-14 provisions for impact load situations. The modification that A23.3-14 makes to ACI 318-14 is how the strength reduction factors are applied to the capacity of an anchor. Despite this change, the magnitude of the anchor capacity reduction is the same for both ACI 318-14 and A23.3-14.

For a post-installed adhesive anchor system to be used in practice it must be qualified under the testing requirements of ACI 355.4 (2011) or both ACI 355.4-11 and AC 308 (ICC, 2013). ACI 355.4-11 prescribes testing requirements for post-installed chemical adhesive anchors, and AC 308 modifies the ACI 355.4-11 requirements for uncoated reinforcing bars post-installed with a chemical adhesive.

Because the use of epoxy-coated reinforcing bars post-installed with a chemical adhesive is relatively new, there are gaps in the literature about how the epoxy coating affects the bond between the reinforcing bars and adhesive. Two studies were found that address epoxy-coated reinforcing bars post-installed with a chemical adhesive (Meline et al., 2006; Dickey, 2011).

Meline et al. (2006) looked at how the epoxy coating affected the bond between post-installed reinforcing bars and an adhesive. The average ratio of the tested ultimate load to the manufacturer published ultimate load was 0.99 for an embedment depth of 9 in. and 1.02 for an embedment depth of 6.75 in. Based on the results, Meline et al. concluded that epoxy-coated reinforcing bars post-installed with a chemical adhesive performed similarly to uncoated reinforcing bars post-installed with a chemical adhesive. While the study by Meline et al. provided valuable knowledge, there were some limitations.

Dickey (2011) tested how the bond between epoxy-coated reinforcing bars post-installed with a chemical adhesive and uncoated reinforcing bars post-installed with a chemical adhesive differ. The loads applied to the reinforcing bars post-installed with a chemical adhesive were impact loads rather than static loads because Dickey was interested in developing a design procedure for concrete traffic barriers attached to bridge decks. However, Dickey performed a static tension test to determine the ratio of the static pullout capacity to the impact pullout capacity. The relationship of the static pullout capacity to the impact pullout capacity was determined to be 1.06, so Dickey concluded that the post-installed reinforcing bars had similar capacity regardless of whether the applied load was impact or static. After testing, Dickey concluded that protective epoxy-coatings on the post-installed reinforcing bars with a chemical adhesive decreased the ultimate bond strength by approximately 9% when impact loads were applied.

The effect an epoxy coating has on the bond strength between cast-in-place reinforcing bars and concrete was investigated because there is a gap in the literature related to how an epoxy-coating affects the bond strength between post-installed reinforcing bars and adhesives. Eligehausen et al. (2012) concluded that the behavior of cast-in-place and post-installed reinforcement were similar because experimental results from each type of reinforcement were within 20% of each other. Mathey and Clifton (1976), Johnston and Zia (1982), and Treece and Jirsa (1989) performed studies which examined the effect an epoxy coating had on the bond strength between cast-in-place reinforcing bars and concrete. All three of the studies found a reduction in the bond strength between the epoxy-coated cast-in-place reinforcing bars and concrete. The magnitude of the bond strength reduction varied between the studies (15 to 32%). ACI 318 (2014) incorporated the results of the three studies in their development length equation with an epoxy-coating modification factor (ψ_e) that ranges from 1.2 to 1.5, based on epoxy-coating, reinforcing bar cover, and reinforcing bar spacing. A value of 1.0 is used for ψ_e when uncoated reinforcing bars are used.

An online survey was conducted to understand the usage of epoxy-coated reinforcing bars post-installed with a chemical adhesive by U.S. state DOTs. The results of the survey indicated that, of the 30 DOTs that responded to the survey, 12 do not use epoxy-coated rebar post-installed with a chemical adhesive. The remaining 18 DOTs used epoxy-coated rebar post-installed with a chemical adhesive. The applications and calculation of bond strength of the DOTs that use epoxy-coated rebar post-installed with a chemical adhesive varied. Some DOTs followed the manufacturer’s literature, some followed ACI 318 (2014) Chapter 17, and some followed their own procedure for bond strength calculations.

Table 2.1 Minimum characteristic bond stresses from ACI 318-14

Installation and service environment	Moisture content of concrete at time of anchor installation	Peak in-service temperature of concrete (˚F)	τ_{cr} (psi)	τ_{uncr} (psi)
Outdoor	Dry to fully saturated	175	200	650
Indoor	Dry	110	300	1,000

Table 2.2 ACI 318-14 strength reduction factors (ϕ) for post-installed anchors

Category	Condition A[1]	Condition B[2]
Category 1 (low sensitivity to installation and service-conditions)	0.75	0.65
Category 2 (medium sensitivity to installation and service-conditions)	0.65	0.55
Category 3 (high sensitivity to installation and service-conditions)	0.55	0.45

[1] Where supplementary reinforcement is provided.
[2] Where supplementary reinforcement is not provided.

Table 2.3 Test program 7a from ACI 355.4-11 Tables 3.1

Test			Assessment			
Test	Purpose	Test	Load and displacement	f'_c[1]	h_{ef}[2]	Minimum sample size, n_{min}
Tension tests in uncracked and cracked concrete	Tension in low-strength concrete	Tension, unconfined, single anchor away from edges[3]	Requirements on coefficient of variation[4], requirements on load-displacement behavior[5], determination of bond stress	Low	Min Max	Five[6]

[1] Low-strength concrete: 2,500-4,000 psi; high-strength concrete: 6,500-8,500 psi.
[2] Where manufacturer specifies multiple embedment depths, test anchor at minimum or maximum embedment depth.
[3] Alternatively, tests may be performed as confined.
[4] Limit of 15% of the coefficient of variation for adhesive anchor testing.
[5] Requires the point at which the adhesive anchor losses initial adhesion to concrete to be greater than 50% of the mean tensile strength of the anchor.
[6] Test all diameters of anchors being evaluated.

Table 2.4 Tension test results for epoxy-coated reinforcing bars from Meline et al. (2006)

Embedment depth (in.)	Epoxy adhesive type	Average ultimate load from testing for epoxy-coated reinforcing bars (lb)	Manufacturer published ultimate load for uncoated reinforcing bars (lb)	Ratio of tested load to manufacturer load	Failure mode (# of tests with this mode)
6.75	Simpson SET22	33,704	33,338	1.01	Adhesive/concrete (3) Adhesive/rebar (2)
	Red Head Epcon	31,950	31,678	1.01	Adhesive/concrete (3) Adhesive/rebar (2)
	CIA-GEL 7000	31,436	30,082	1.05	Adhesive/concrete (5)
	Average of ratios for tested load to manufacturer load			1.02	-
9	Simpson SET22	40,098	40,926	0.98	Adhesive/rebar bond (2) Chuck slip (1) Rebar yielding (2)
	Red Head Epcon	34,904	33,466	1.04	Adhesive/concrete (2) Adhesive/rebar (3)
	CIA-GEL 7000	37,786	40,318	0.94	Adhesive/concrete (4) Rebar yield (1)
	Average of ratios for tested load to manufacturer load			0.99	-

Table 2.5 Results from impact tensile tests on single epoxy-coated and uncoated reinforcing bars from Dickey (2011)

Test number	Reinforcing bar coating	Reinforcing bar size	Tested ultimate load (kips)	Ratio of tested load for epoxy-coated to average uncoated	Failure mode
1	Uncoated	#5	38.4	-	Steel fracture
2	Uncoated	#5	39.4	-	Steel fracture
		Average uncoated	*38.9*		-
3	Epoxy-coated	#5	35.0	0.90	Steel fracture
4	Epoxy-coated	#5	35.9	0.92	Steel fracture
5	Uncoated	#6	47.2	-	Steel/adhesive bond
		Average uncoated	*47.2*		-
6	Epoxy-coated	#6	41.3	0.87	Concrete/adhesive bond
7	Epoxy-coated	#6	43.7	0.93	Steel/adhesive bond
			Average ratio of tested ultimate load for epoxy-coated to uncoated #5 and #6 bars	*0.90*	-

Table 2.6 Comparison of experimental column-foundation moment capacity for specimens with cast-in-place (CIP) and post-installed (PI) reinforcing bars from Eligehausen et al. (2012)

	Specimen number	Number and size of reinforcing bar	Embedment depth (in.)	Experimental moment capacity (kip-ft)
CIP reinforcement	1	(4) #5	9.4	56.2
	2	(2) #10	16.5	131.1
			Average	*93.7*
PI reinforcement	3	(4) #5	9.4	89.4
	4	(2) #10	16.5	137.2
			Average	*113.3*
			Ratio of average moment capacity of post-installed reinforcement to cast-in-place reinforcement	*1.2*

Table 2.7 Development length modification factors for coating of reinforcing bars (ψ_e)

Condition	Value of factor
Epoxy-coated reinforcement with a clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
Epoxy-coated reinforcement for all other conditions	1.2
Uncoated reinforcement	1.0

Table 2.8 Departments of transportation that do not use epoxy-coated rebar post-installed with a chemical adhesive (12 of 30 respondents)

Alabama	Colorado	Florida	Louisiana
Maryland	Mississippi	Missouri	New Hampshire
New Mexico	Tennessee	Texas	Virginia

Table 2.9 Departments of transportation that use epoxy-coated rebar post-installed with a chemical adhesive (18 of 30 respondents)

Agency	Procedure to calculate bond strength	Applications
Anonymous 1	Manufacturer's literature	-
Anonymous 2	Manufacturer's literature	-
Anonymous 3	ACI 318-14 Chapter 17	Chain link fence and bridge railing
Arkansas	Manufacturer's literature	Bridge widening jobs or replacing damaged rebar
California	Other process	Corrosive environments
Delaware	Manufacturer's literature	Widening or extending an existing concrete element (applications with no tension)
Georgia	Manufacturer's literature	Dowel reinforcement in sidewalks and other various applications. Not allowed in applications where impact loads cause tension and in overhead applications that cause sustained tension.
Iowa	Other process	Retrofit barrier rails and miscellaneous concrete repair
Kentucky	ACI 318-14 Chapter 17 and manufacturer's literature	Rehab work and phased construction
Michigan	ACI 318-14 Chapter 17, manufacturer's literature, and other process	Substructure widening (footing/column/pier cap), bridge barrier, temporary works, lane ties, and other miscellaneous applications
Minnesota	ACI 318-14 Chapter 17 and other process	Attaching barriers to decks, retrofits to bridge end blocks, and other situations in which only shear and unsustained tension are being transferred
New York	Manufacturer's literature	Adding concrete to the face of a bridge abutment or similar applications. Not allowed in sustained tension
North Dakota	Manufacturer's literature	Approach slab lip repair and attaching barriers to decks
Ohio	Other process	Non-structural applications
Pennsylvania	ACI 318-14 Chapter 17	Pier column jacketing and other cases where the dowel bar is subject to shear and no direct tension
Utah	Manufacturer's literature	Modifications are made to an existing structure
Wisconsin	ACI 318-14 Chapter 17	Wing repairs, abutment paving blocks, abutment back walls, and some parapet anchorage
Wyoming	Manufacturer's literature	Curb tie bar into bridge deck

Notes:

11 agencies used the manufacturer's literature to calculate bond strength
6 agencies used ACI 318-14 Chapter 17 to calculate bond strength
5 agencies used an other process to calculate bond strength

Table 2.10 Verification procedures used by departments of transportation that use rebar post-installed with a chemical adhesive (11 of 30 respondents)

Agency	Types of verification tests used	Frequency and applied load of verification tests
Anonymous 1	-	-
Arkansas	Field testing as necessary	Test to 125% of anchor yield strength
California	Inspection at installation. Material tests on the adhesives	-
Delaware	Inspection during installation	-
Georgia	-	-
Iowa	Office of Construction and Materials may sample and test to verify compliance with specifications	-
Kentucky	Proof test on certain jobs	Test to yield on about 10% of all installed anchors
Michigan	Proof tests to verify that the product on the qualified product list is still performing as intended. Field tests to verify contractor installation	Proof tests are to 125% of anchor yield strength. Field test are to 90% of anchor yield strength, and 3 tests are performed
Minnesota	Proof testing	Test 1% of anchorages, no less than 8. The proof load does not exceed 2.2 kips
New York	Have an inspector present during installation to verify the process. Inspector and installer must be certified	-
North Dakota	-	-
Ohio	-	-
Pennsylvania	Proof test per ASTM E488 and E1512	Test 10% of anchors and test to 85% of anchor tensile capacity
Utah	In-situ static tension test on project specific basis	Anchors tested to 90% of yield strength
Wisconsin	Installation by or under the direct supervision of an ACI/CRSI certified installer and/or field-verified by non-destructive pullout testing according to ASTM E488	Proof test the first 3 anchors of each rebar size installed, then one or 5% of the remaining anchors, whichever is greater. Proof load to 80% of the anchor yield strength unless the plans show otherwise
Wyoming	-	-

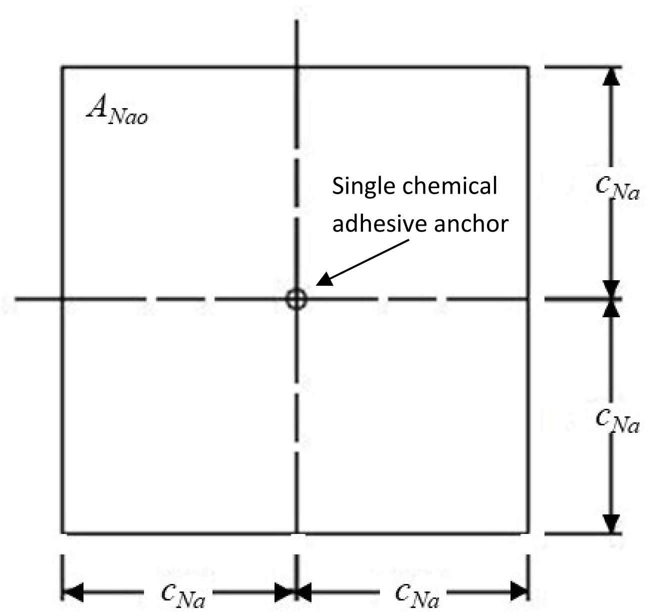


Figure 2.1 Geometry of a single chemical adhesive anchor away from any concrete edges

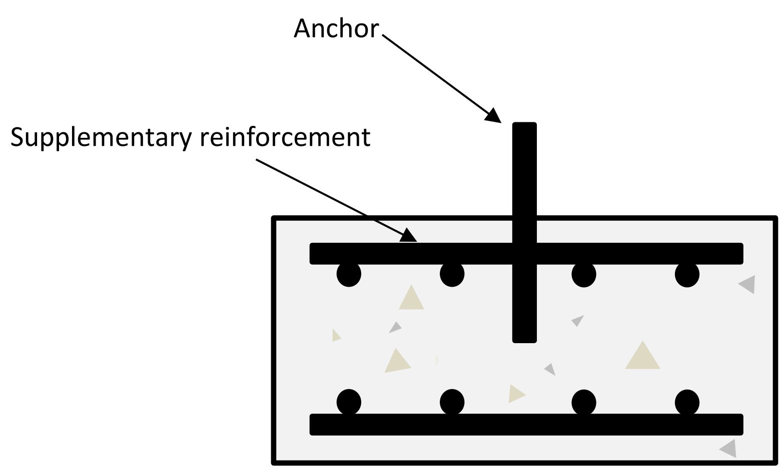


Figure 2.2 Example of supplementary reinforcement

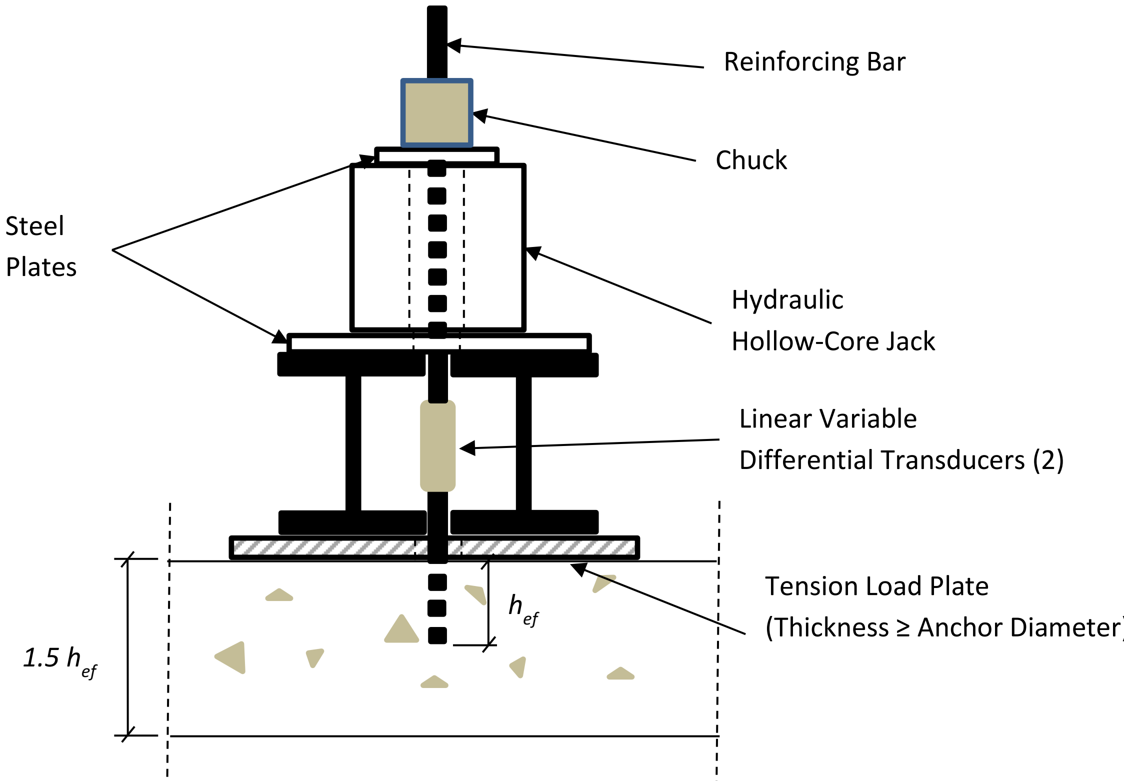


Figure 2.3 Example test setup for ACI 355.4-11 tension test 7a in accordance with ASTM E488 (2015)

CHAPTER 3: LABORATORY EXPERIMENTAL PROGRAM, SLAB DETAILS, AND TESTING PROCEDURE

3.1 INTRODUCTION

To achieve the project objectives, a laboratory study was conducted using two concrete slabs to compare the tensile pullout strength of epoxy-coated and uncoated reinforcing bars. One slab contained epoxy-coated reinforcing bars and the other contained uncoated reinforcing bars. The test setup and specimens were built and tested in accordance with ACI 355.4 (2011), ASTM E488 (2015) and the chemical adhesive manufacturer's literature. The amount of applied load, amount of displacement, and the failure mode were recorded during each test. All of the tests were conducted in uncracked concrete. Tests were conducted to capture the confined tensile pullout strength of post-installed reinforcing bars away from concrete edges in accordance with test number 7a in ACI 355.4-11 (2011).

3.2 LABORATORY EXPERIMENTAL PROGRAM

A total of 48 tests were conducted to investigate the effect of the specified concrete strength (4,000 psi), reinforcing bar size (#5), embedment depth (5 in. or $8d_b$), reinforcing bar type (epoxy-coated and uncoated), and type of chemical adhesive. The reinforcing bar embedment depth was not the manufacturer's minimum or maximum as specified in ACI 355.4 (2011). An embedment depth of 5 in. was chosen because it is common practice for MnDOT to post-install #5 reinforcing bars with a chemical adhesive at that embedment depth. Four different chemical adhesives were used: Powers AC100+ Gold, Red Head A7+, Hilti HIT-RE 500 V3, and ATC Ultrabond 365CC. The variables investigated in the laboratory study can be seen in Table 3.1.

3.3 SLAB DESIGN AND FABRICATION

This section provides details about the design and fabrication of the two slabs where reinforcing bars were post-installed with chemical adhesives. Furthermore, details about the installation of the post-installed reinforcing bars and the material properties of the concrete and reinforcing bars are discussed.

3.3.1 Slab Design Details

Two slabs were constructed. One slab contained epoxy-coated reinforcing bars post-installed with chemical adhesives and the other slab contained uncoated reinforcing bars post-installed with the same chemical adhesives. Both of the slabs were unreinforced concrete, which means they were cast without any supplemental reinforcement. The slab dimensions were 6 ft 2 in. wide, 11 ft 6 in. long, and 8 in. deep. The depth of the slab was chosen based on guidance from ASTM E488 (2015) and the chemical adhesive manufacturer's literature. ASTM E488 noted that the concrete member thickness shall be at least one and a half times the embedment depth of the post-installed reinforcing bar ($t_{slab} \geq 1.5h_{ef}$). The chemical adhesive manufacturer's literature specified the required minimum slab thickness as the embedment depth plus a clear cover distance that was different for each manufacture. The embedment depth used was 5 in. The controlling slab thickness was 7.5 in. based on ASTM E488 rather than the manufacturer's literature because the requirements of ASTM E488 were more conservative. A slab thickness of 8 in. was used to make formwork construction simple. For a confined tensile pullout strength test away from concrete edges (ACI 355.4-11 test 7a), there was no guidance for the dimensions of the slab, thickness of the slab, or the spacing between post-installed reinforcing bars in ACI 355.4 (2011). Additionally, there was no guidance in ASTM E488 for the dimensions of the slab or the spacing between post-installed reinforcing bars. The slab dimensions and post-installed reinforcing bar spacing were chosen to provide adequate room to set up the tensile pullout test jig, which was 16 in. wide. Spacing and dimension requirements from ACI 318 (2014) for concrete breakout and bond strength group effects were considered but did not control. A 9 in. spacing between the edge of the slab and the first post-installed reinforcing bar was provided so that the tensile pullout test jig would fully contact concrete (9 in. > half of the tensile pullout test jig width).

The dimensions and volume (47.27 ft^3 each) of the slabs were large and no reinforcement was provided. Large masses of unreinforced concrete have a tendency to shrink and crack over time (ACI 224R, 2001; Carlson et al., 1979). Measures were taken to ensure shrinkage cracking did not occur at the post-installed reinforcing bar locations. Two perpendicular joints were added to help control the location of potential cracks and the slab was cast on plastic sheeting to allow for concrete movement over time (i.e., expansion or shrinkage). One joint ran parallel to the 11 ft 6 in. dimension and divided the slab in two equal halves, and the other joint ran parallel to the 6 ft 2 in. dimension and divided the slab into approximately two equal halves. The control joint configuration was selected to avoid the location of the post-installed reinforcing bars. The dimensions for the two slabs were the same and are shown in Figure 3.1.

3.3.2 Slab Fabrication

Slab formwork consisted of a 0.5 in. thick plywood base and side walls built with 1.5 in. by 8 in. lumber. The side wall height corresponded to the selected 8 in. slab depth. The lumber side walls were attached to the plywood base with 45˚ kicker supports spaced at 18 in. on center, one 2 in. long screw, and subfloor adhesive. Additional blocks of wood were placed between each 45˚ kicker support at a spacing of 18 in. on center to provide extra restraint against formwork blowout due to the selfweight of concrete. Slab formwork can be seen in Figure 3.2.

The two slabs were cast on April 5, 2018 in the University of Minnesota Duluth (UMD) Structures Laboratory. Since the slabs were unreinforced concrete, a slump measurement was not checked because workability was not a concern. Four and a half cubic yards of concrete was ordered from Arrowhead Concrete Works, Inc. for the two slabs and for 18 concrete compressive strength cylinders (4 by 8 in.).

Concrete was poured from the truck into a 0.75 yd^3 concrete bucket, which was attached to the overhead crane and transported to where the slabs were being cast in the UMD Structures Laboratory. The concrete was consolidated using a handheld wand vibrator. Care was taken to ensure that the top-of-slab surface was level and well finished for seating of the tensile pullout test jig. ASTM 488 (2015) noted that the finished concrete surface of a test member shall be a formed or steel-troweled finish unless otherwise specified by the chemical adhesive manufacturer's literature. A level top-of-slab surface was achieved by screeding off excess concrete, using a bull float (Figure 3.2), and hand finishing the concrete. The hand finishing processes consisted four steps. First an edger was used around the slab perimeter to provide a clean edge and help keep the formwork from restraining shrinkage (i.e., curing cracks). Second, a joint tool was used to add the control joints to the slabs. Third, a magnesium concrete hand float was used to smooth any unevenness caused from the edging and joint tools. Finally, a steel concrete hand float was used to give the concrete a smooth final finish.

The slabs were subjected to a 7-day continuous moist cure by covering the specimens with water saturated burlap and plastic. The 4 by 8 in. cylinders were cured under the plastic with the slab to ensure consistency. The formwork was stripped after 24 hours (April 6, 2018). A timeline of the slab fabrication and testing is shown in Table 3.2.

3.3.3 Installation of the Post-Installed Reinforcing Bars

Installation of the post-installed reinforcing bars was completed by a MnDOT approved contractor. There were several reasons for having a contractor do the installation. First, the contractor was a certified ACI adhesive anchor installer. Second, it was important to collect results that were representative of field conditions. Third, use of an experienced contractor likely provided consistency in the drilled hole depth and vertical reinforcing bar alignment. PCiRoads, LLC was the contractor that completed the installation of post-installed reinforcing bars, which included drilling the holes, cleaning the drilled holes, filling the drilled holes with the four different chemical adhesives, and inserting the reinforcing bars into the filled holes.

To drill the holes, a hammer drill was used with a 0.75 in. diameter drill bit and a Bosch HDC200 dust collection system. A 0.75 in. dimeter drill bit was specified by all four of the chemical adhesive manufactures for installation of #5 reinforcing bars. To get an embedment depth of 5 in., the drill bit was marked using duct tape. The hammer drill, drill bit, and drilling process are illustrated in Figure 3.3 and Figure 3.4. A compressed air nozzle, a wire brush attached to a drill, and a dust collection system were used to clean the holes after drilling. The hole cleaning process used in the laboratory was the same process PCiRoads follows in the field, which the contractor said was taught during the ACI adhesive anchor certification training. The hole cleaning process that was used also followed the process specified in the chemical adhesive manufacturer's literature for the four different chemical adhesives. The holes were first blown clean using compressed air, starting at the bottom of the hole and working up to the top. Then the holes were brushed for three seconds using a wire brush attached to a drill, starting at the bottom of the hole and working up to the top. Figure 3.5 shows the compressed air nozzle and the wire brush attached to a drill. The process of blowing out and brushing the holes was done three times per hole and is shown in Figure 3.6. The difference between a clean hole and a hole that had not yet been cleaned is shown in Figure 3.7.

After the holes were cleaned, reinforcing bars were installed with the four different chemical adhesives: Powers AC100+ Gold, Red Head A7+, Hilti HIT-RE 500 V3, and ATC Ultrabond 365CC. All of the adhesives were two-part chemical adhesives, which means a mixing nozzle used during installation automatically mixes the two-part chemical adhesive as it is being dispensed out of the tool into the hole. The four chemical adhesives and their respective dispensing tools are shown in Figure 3.8. To ensure proper mixing, the first two pumps from a new tube of chemical adhesive were not used as shown in Figure 3.9. The holes were filled two thirds full with chemical adhesive starting from the bottom and moving upwards. This process was followed to ensure there were no air voids surrounding the post-installed reinforcing bar.

The reinforcing bars were inserted using a twisting motion to ensure that the chemical adhesive coated the entire surface area of the reinforcing bar. Care was taken to ensure that the reinforcing bars stayed vertical and a wooded brace was attached to some of the reinforcing bars to provide stability as shown in Figure 3.10. The location of the epoxy-coated and uncoated reinforcing bars and the type of chemical adhesive used to install each row of bars is shown in Figure 3.11. One row of each slab was left unused to provide space if additional tensile pullout strength tests needed to be performed.

3.3.4 Material Properties

The steel reinforcing bars conformed to ASTM A615 (2016). The epoxy-coating conformed to ASTM A775 (2017). Both the epoxy-coated and uncoated reinforcing bars were donated by ABC Coating Co. The nominal and manufacture provided steel material properties for the epoxy-coated and uncoated reinforcing bars are shown in Table 3.3.

The chemical adhesive properties were proprietary and varied for each manufacturer. Powers AC100+ Gold was a vinylester chemical adhesive. Red Head A7+ was a hybrid chemical adhesive. Hilti HIT-RE 500 V3 was an epoxy mortar chemical adhesive. ATC Ultrabond 365CC was a cement and silane hybrid chemical adhesive. All of the chemical adhesives were donated by their respective manufacturers. The manufacturers for the Powers AC100+ Gold, Red Head A7+, and Hilti HIT-RE 500 V3 also donated the dispensing tools for their adhesives. The different chemical adhesive manufacturer's literature provided values for the uncracked and cracked bond strength (τ_{uncr} and τ_{cr}). However, in the Appendix A example design calculations for a retrofit bridge traffic barrier and a retrofit crash strut for a two-column bridge pier, an uncracked bond strength of 1,000 psi and a cracked bond strength of 500 psi were used (MnDOT, 2016).

The concrete mix design was MnDOT 3Y33. The maximum aggregate size for the concrete mix was 0.75 in., the specified minimum 28-day compressive strength (f'_c) was 4,000 psi. The concrete mix had a specified slump of 3 in. The material properties for the concrete mix are provided in Table 3.4. The 28-day, start-of-testing, and end-of-testing concrete compressive strength results shown in Table 3.5.

3.4 TEST PROCEDURE, SETUP, AND INSTRUMENTATION

This section provides details about the test procedure, laboratory experimental setup, and instrumentation used during the confined tensile pullout strength testing of the post-installed reinforcing bars.

3.4.1 Test Procedure

The procedure for the tensile pullout strength test of the post-installed reinforcing bars followed guidance from ACI 355.4 (2011) and ASTM E488 (2015). ACI 355.4 is the standard used to qualify post-installed chemical adhesive anchor systems for use in practice and ASTM E488 provides test procedures to determine the tensile strength of post-installed reinforcing bars.

ACI 355.4 Table 2.3 presented test details to obtain the tensile pullout strength of a post-installed reinforcing bar away from concrete edges in test number 7a. Test 7a can either be performed as an unconfined or confined test, and tests in this research project were conducted as confined tests. Requirements for ACI 355.4 test 7a included:

1. Test a single post-installed reinforcing bar
2. Post-install the reinforcing bars away from concrete edges
3. The member concrete compressive strength should be between 2,500 and 4,000 psi
4. Test either the minimum or maximum embedment depth
5. The minimum number of tensile pullout tests is five

Most of the requirements from ACI 355.4 for confined tensile pullout strength testing of the post-installed reinforcing bars away from concrete edges were followed in this research. Requirement three from ACI 355.4 test 7a was not met in this project. It is common practice for MnDOT to post-install reinforcing bars with a chemical adhesive into 3Y33 bridge deck concrete mixtures, which has a minimum 28-day compressive strength of 4,000 psi. In reality, the *in-situ* concrete compressive strength frequently exceeds the specified strength. That was also the case for the slabs in this project, which had an average strength of 5,151 psi at the time of testing as shown in Table 3.5. Requirement four was not met in this project because it is common practice for MnDOT to post-install #5 reinforcing bars with a chemical adhesive at an embedment depth of 5 in. Pullout tests were conducted at the embedment depth of 5 in. for each chemical adhesive and each reinforcing bar coating, as outlined in Table 3.1.

While ACI 355.4 (2011) presented the tensile pullout strength test guidelines, ACI 355.4 did not provide guidance on how to perform the test. ASTM E488 (2015) provided test requirements and procedures to determine the tensile strength of post-installed reinforcing bars. The requirements included tolerances for the load cells used to measure load (measure accurately to ±1% of the anticipated ultimate load), the linear variable differential transducers (LVDTs) used to measure displacement (measure accurately to ±0.001 in.), and how often the load and displacement should be recorded (once per second or 1 Hz). The testing procedures from ASTM E488 included placing the tensile pullout test jig on a sheet of polytetrafluoroethylene (PTFE) or other friction-limiting material with a minimum thickness of 0.020 in., centering the tensile pullout test jig over the post-installed reinforcing bar, centering the LVDTs on the post-installed reinforcing bar, applying an initial load up to 5% of the estimated maximum load capacity, and increasing the load so that peak load occurs after one to three minutes.

3.4.2 Test Setup

ASTM E488 (2015) provided requirements for the test jig used to evaluate tensile pullout strength of post-installed reinforcing bars. The requirements for the tensile pullout test jig included having sufficient capacity to prevent yielding of all steel components used in the jig, a specific load plate thickness, and a specific confining hole diameter. An analysis of all of the steel components used in the tensile pullout test jig was done using the American Institute of Steel Construction (AISC) Steel Construction Manual (2011), and no steel yielding was expected under the anticipated loads. ASTM E488 specified that the load plate thickness should be greater than or equal to the diameter of the post-installed reinforcing bar ($t_{plate} \geq d_{bar}$) and the load plate confining hole diameter should be between one and a half and two times the drilled hole diameter ($1.5d_{hole}$ and $2.0d_{hole}$). The diameter of the hole drilled for the post-installed #5 reinforcing bars was 0.75 in. and a confining hole diameter of 1.375 in. was used ($1.83d_{hole}$).

The tensile pullout test jig had a top and bottom load plate that consisted of a 0.625 in. thick A36 steel rectangular plate that was 10 by 16 in. The dimensions of the bottom plate were chosen to prevent concrete breakout calculated using ACI 318 (2014) Chapter 17. The concrete breakout cone projects out one and a half times the embedment depth ($1.5h_{ef}$ = 7.5 in.), and the bottom load plate dimensions were chosen to be larger than the concrete breakout cone to prevent its formation. The top load plate was selected to match the dimensions of the bottom plate and was adequate to support the hydraulic cylinder. Two W8x31 A992 steel sections were used to provide space to attach LVDTs to the reinforcing bar for displacement measurement. The W8x31 sections were 10 in. long. Two LVDTs were attached to the reinforcing bar using an LVDT jig, which consisted of a 1 by 1 in. steel angle welded to a rebar splice coupler as shown in Figure 3.12. A SPXflow RH1003 hydraulic cylinder was used to pull on the reinforcing bar, and a WB RS #5 wedge chuck was used to grip the reinforcing bar. The hydraulic cylinder was attached to a Simplex P82A hydraulic hand pump. The tensile pullout test jig can be seen in Figure 3.13.

3.4.3 Instrumentation and Data Acquisition

Two different types of instrumentation were used during testing. The first was a pair of TransTek Series 240, Model 0243-0000 LVDTs and the second was an Omega PX603-10KG5V in-line digital pressure transducer. The LVDTs were attached to the reinforcing bar using the LVDT jig and measured the deflection during pullout of the post-installed reinforcing bar to an accuracy of 0.5% of the actual displacement with a stroke of 1 in. The two LVDTs can be seen in Figure 3.14. The digital pressure transducer could measure pressure up to 10,000 psi and had an accuracy of 0.4% of the actual pressure. The digital pressure transducer, which was mounted in-line between the hydraulic hand pump and the hydraulic cylinder, can be seen in Figure 3.15.

Data from each test were collected with data acquisition software installed on the laboratory computer. Data were collected at a sample rate of five data points per second (5 Hz) for each test. Prior to testing, all instrumentation was calibrated using the manufacturer’s instructions. The failure mechanism for each test specimen was documented by taking pictures. The files from the data acquisition software were converted into spreadsheet files for further analysis following the completion of each test.

Table 3.1 Variables investigated in the laboratory experimental program

Set	Concrete strength (psi)	Rebar size	Embedment depth (in.)	Reinforcing bar type	ACI 355.4-11 test and reference	Epoxy adhesive type	Number of tests
1	4,000	5 (8d_b)	5	Uncoated	Tension – 7a*	Powers AC100+ Gold	6
						Red Head A7+	6
						Hilti HIT-RE 500 V3	6
						ATC Ultrabond 365CC	6
2	4,000	5 (8d_b)	5	Epoxy-coated	Tension – 7a*	Powers AC100+ Gold	6
						Red Head A7+	6
						Hilti HIT-RE 500 V3	6
						ATC Ultrabond 365CC	6
						Sum	*48*

* Tension 7a tests were be conducted as confined.

Table 3.2 Laboratory slab fabrication and testing timeline

Date	Time	Time since casting (days)	Event
April 5, 2018	9 am – 11 am	-	Slab concrete placed
	11 am – 1 pm		Slab hand finished
	3 pm		Slab covered with pre-soaked burlap and plastic
April 6, 2018	5 pm	1	Slab formwork removed
April 12, 2018	1 pm	7	Slab burlap and plastic removed
May 3, 2018	2 pm	28	Slab 28-day concrete compressive strength test
May 17, 2018	10 am – 1 pm	42	Installation of post-installed reinforcing bars
May 21, 2018	-	46	Start of tensile pullout testing
	-		Slab concrete compressive strength test
June 6, 2018	-	69	Tensile pullout testing concluded
	-		Slab concrete compressive strength test

Table 3.3 Nominal and manufacturer provided steel material properties for the uncoated and epoxy-coated reinforcing bars supplied by ABC Coating Co. and manufactured by Nucor, Inc.

	Nominal	Manufacturer Provided	
	Uncoated and Epoxy-coated steel reinforcing bar	Uncoated steel reinforcing bar	Epoxy-coated steel reinforcing bar
Diameter of #5 reinforcing bar (in.)	0.625	0.625	0.625
Area of #5 reinforcing bar (in.^{2})	0.31	0.31	0.31
Yield strength (ksi)	60.00	74.98	70.74
Tensile strength (ksi)	90.00	111.18	108.31
Elongation at ultimate tensile strength (%)	9.0	12.4	12.6
Modulus of elasticity (ksi)	29,000	29,000	29,000

Table 3.4 Slab concrete mix design from Arrowhead Concrete Works, Inc.

Component	Quantity (per yd^3)
Sand	1,235 lb
0.75 in. aggregate	1,830 lb
Cement (c)	533 lb
Flyash	94 lb
Water (w)	32.8 gal
Air entrainer (BASF MasterAir AE90)	6.0 oz
W/C ratio	0.437

Table 3.5 Slab concrete 28-day, start-of-testing, and end-of-testing compressive strength results

Time	Slab 1 epoxy-coated reinforcing bars (psi)	Slab 2 uncoated reinforcing bars (psi)	Average (psi)
28-day	5,260	5,110	5,190
Start-of-testing (48-day)	5,320	5,170	5,240
End-of-testing (69-day)	5,100	5,020	5,060
Average during testing	5,210	5,090	5,150

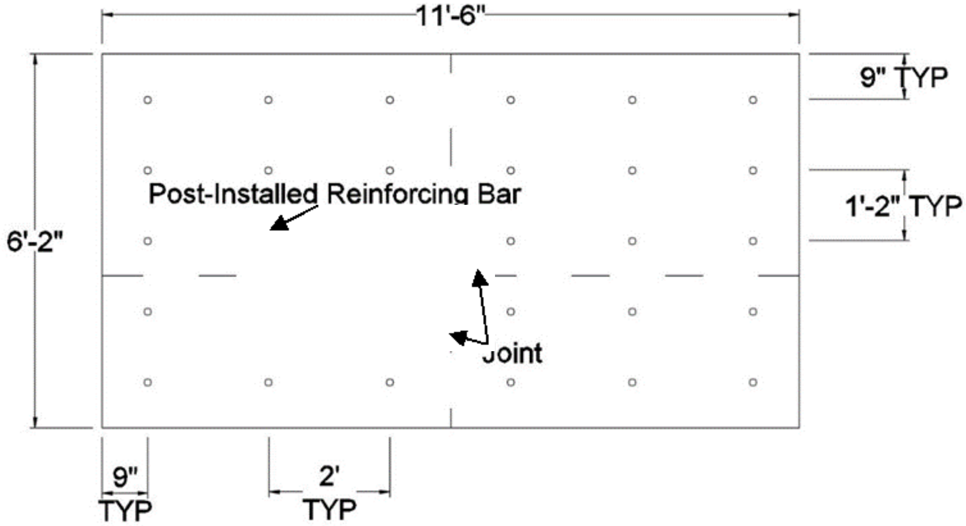


Figure 3.1 Slab dimensions, control joint locations, and spacing of post-installed reinforcing bars away from slab edges

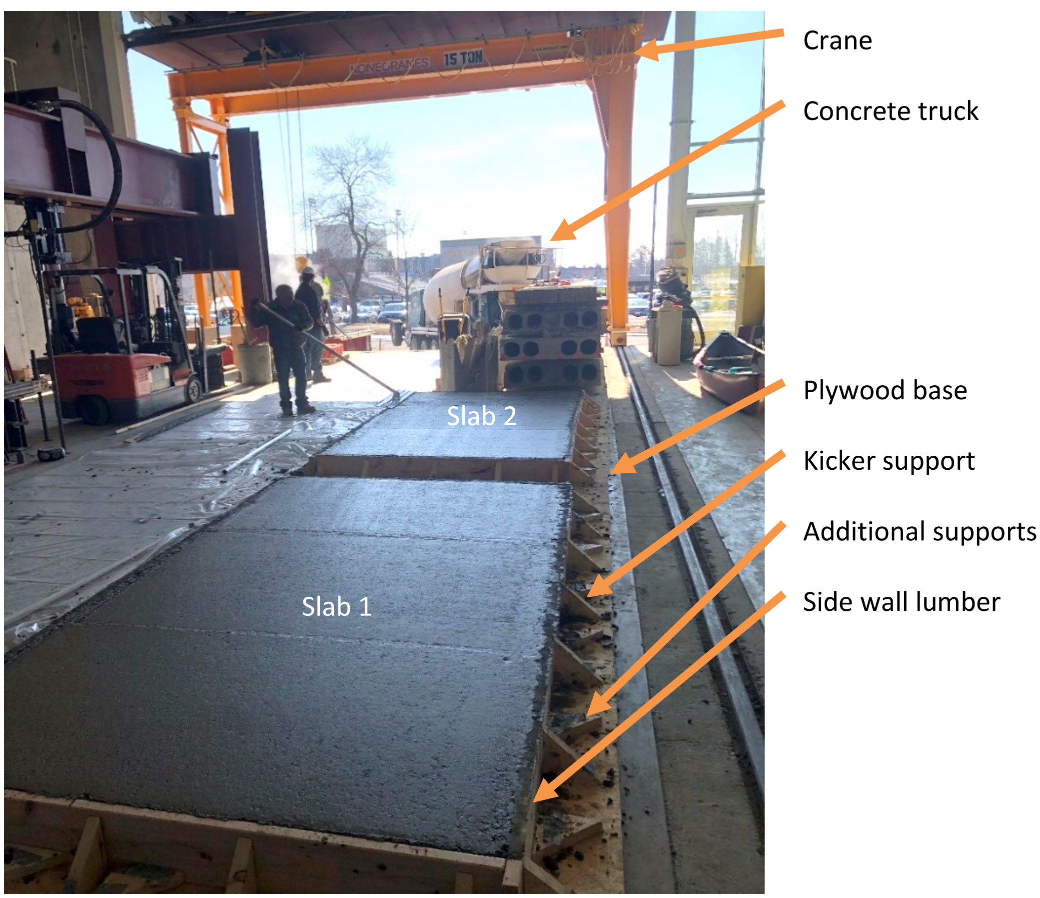


Figure 3.2 Slab location in laboratory, formwork details, and steel-troweled surface finish of slabs

Figure 3.3 Hammer drill and 0.75 in. drill bit marked to a drill depth of 5 in. used to drill the holes for post-installing #5 reinforcing bars

Figure 3.4 Hammer drilling into the slab with duct collecting system in operation

Figure 3.5 Compressed air nozzle and drill mounted wire brush used for hole cleaning

Figure 3.6 Hole cleaning process

Figure 3.7 Difference between clean and unclean holes

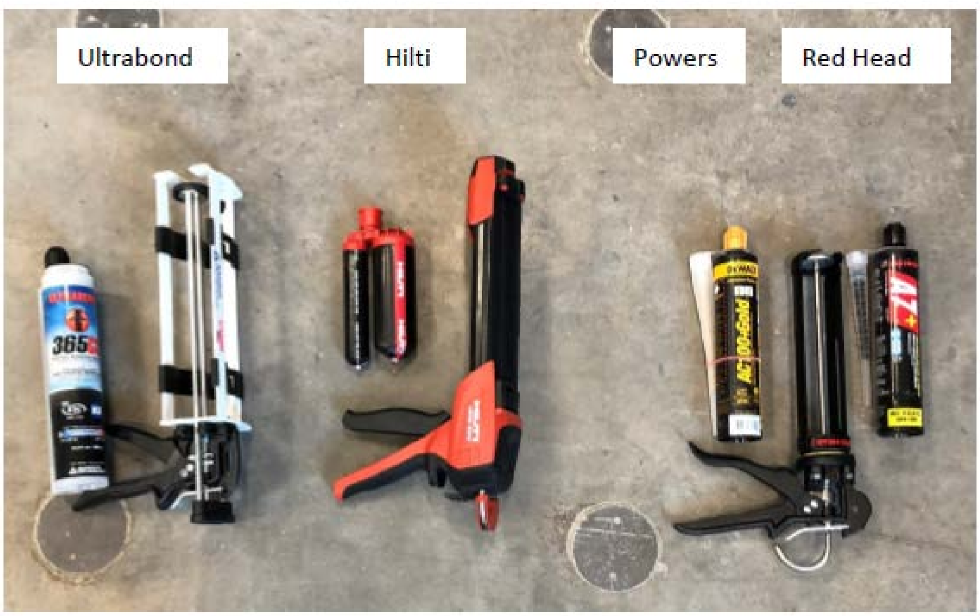


Figure 3.8 Chemical adhesives and dispensing tools used to post-install #5 reinforcing bars

Figure 3.9 Example nozzle used to mix a two-part chemical adhesive and the first two pumps of unused, discarded chemical adhesive

Figure 3.10 Wooden brace attached to some of the reinforcing bars to provide stability and ensure a vertical orientation

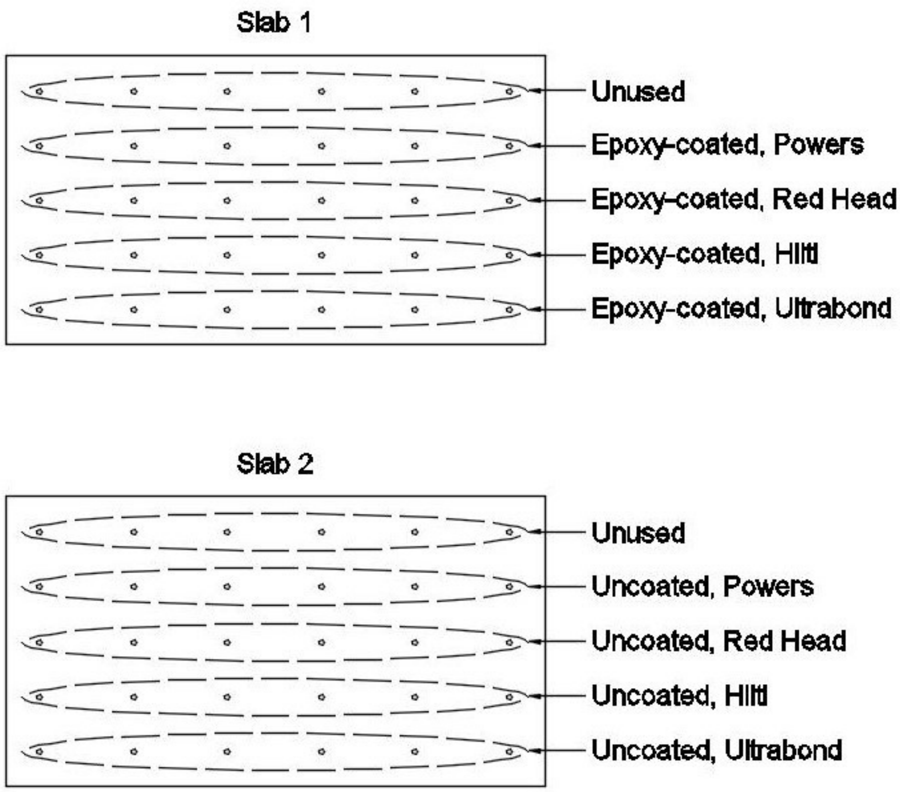


Figure 3.11 Location of epoxy-coated or uncoated reinforcing bars and the type of chemical adhesive used to install each row of bars in the two slabs

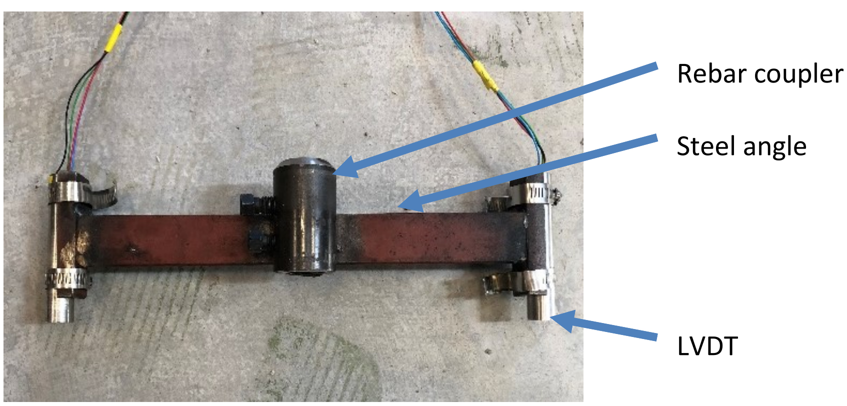


Figure 3.12 LDVT jig that can be attached to a post-installed #5 reinforcing bar to measure vertical displacement during pullout testing

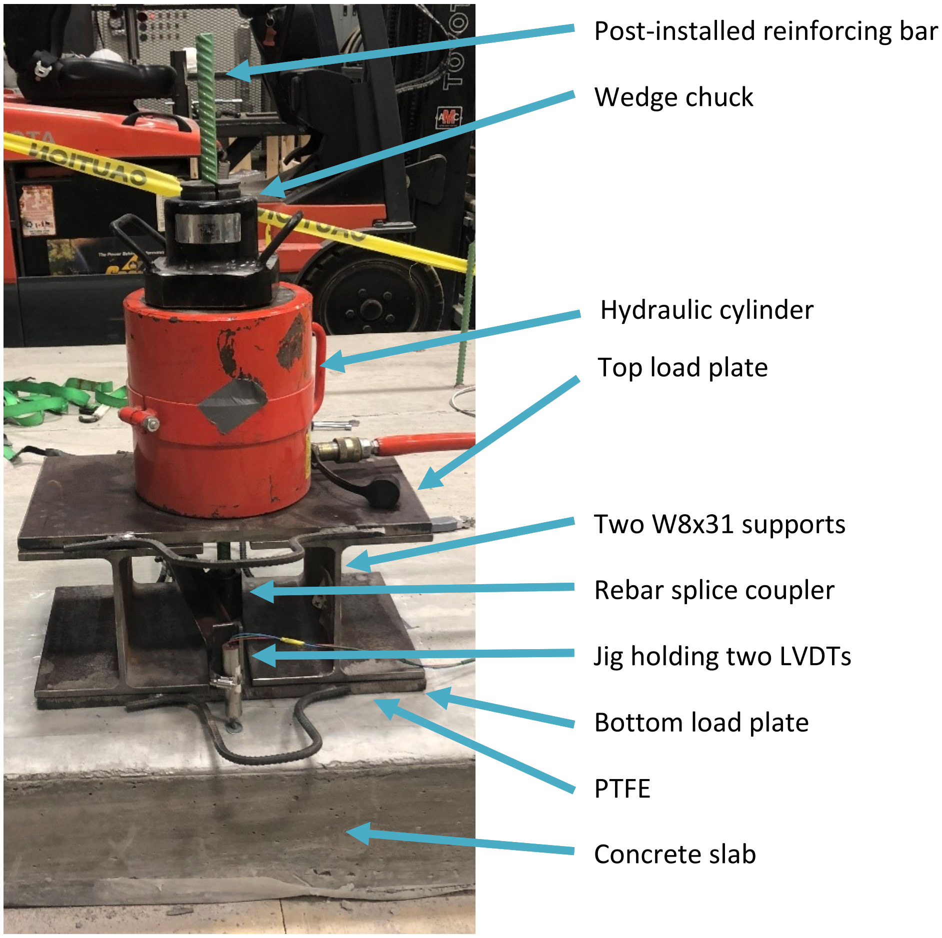


Figure 3.13 Tensile pullout test jig constructed in accordance with ASTM E488 (2015) and used to pull out uncoated and epoxy-coated #5 reinforcing bars embedded 5 in.

Figure 3.14 Linear variable differential transducers (LVDT) with a 1 in. stroke

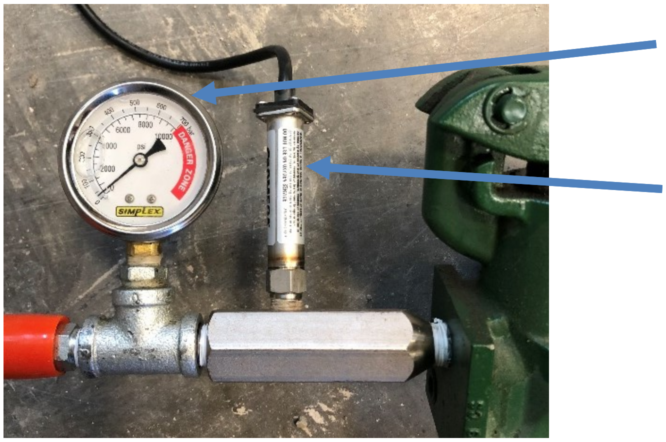


Figure 3.15 In-line pressure dial gauge and digital pressure transducer

CHAPTER 4: RESULTS AND DISCUSSION OF LABORATORY EXPERIMENTAL PROGRAM

4.1 INTRODUCTION

A series of 48 pullout tests were conducted on post-installed reinforcing bars to determine the confined pullout tensile strength in accordance with test number 7a from ACI 355.4 (2011). Half of the tensile pullout strength tests were performed on epoxy-coated reinforcing bars and half were performed on uncoated reinforcing bars. Four different chemical adhesives (Powers AC100+ Gold, Red Head A7+, Hilti HIT-RE 500 V3, and ATC Ultrabond 365CC) were used to post-install both the epoxy-coated and uncoated reinforcing bars. Six tensile pullout strength tests were performed for each combination reinforcing bar coating chemical adhesive.

Each specimen was labeled. Epoxy-coated or uncoated reinforcing bars were labeled with "EC" or "UC," respectively. The type of chemical adhesive was also labeled. When a specimen was installed using Powers AC100+ Gold, it was denoted as "P"; Red Head A7+ was denoted as "R"; Hilti HIT-RE 500 V3 was denoted "H"; finally, ATC Ultrabond 365CC was denoted "U." The six specimens for each group of reinforcing bar coating and chemical adhesive were also labeled with a "1" through "6." For example, the first test of an epoxy-coated reinforcing bar post-installed using Powers AC100+ Gold was labeled "ECP1."

4.2 RESULTS

In this section, the results are categorized by the type of chemical adhesive used. Results are presented showing the differences between the ultimate tensile pullout strength of epoxy-coated and uncoated reinforcing bars; the difference between the ultimate tensile pullout strength of epoxy-coated reinforcing bars and the manufacturer published bond strength with uncoated reinforcing bars; and the difference between the ultimate tensile pullout strength of epoxy-coated reinforcing bars and the MnDOT-specified bond strength with uncoated reinforcing bars. The measured displacement at the ultimate tensile pullout strength for each test is presented. The failure modes for each test are also discussed in this section.

4.2.1 Powers AC100+ Gold

The average ultimate tensile load for epoxy-coated reinforcing bars post-installed using Powers AC100+ Gold was 29.81 kips. The average ultimate tensile load for uncoated reinforcing bars post-installed using Powers AC100+ Gold was 31.55 kips (6% higher). The ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars to the uncoated reinforcing bars was 0.94. The average displacement at the ultimate tensile load for epoxy-coated reinforcing bars was 0.11 in. and the average displacement at the ultimate tensile load for uncoated reinforcing bars was 0.29 in. The experimental data from testing both the epoxy-coated and uncoated reinforcing bars is shown in Table 4.1. The failure mode for all of the epoxy-coated reinforcing bars was bond failure between the concrete and the chemical adhesive. The failure mode for all of the uncoated reinforcing bars was bond failure between the steel and the chemical adhesive. All the failure modes for reinforcing bars installed with Powers AC100+ Gold are shown in Figure 4.1.

4.2.2 RED HEAD A7+

The average ultimate tensile load for epoxy-coated reinforcing bars post-installed using Red Head A7+ was 30.32 kips. The average ultimate tensile load for uncoated reinforcing bars post-installed using Red Head A7+ was 31.27 kips (3% higher). The ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars to the uncoated reinforcing bars was 0.97. The average displacement at the ultimate tensile load for epoxy-coated reinforcing bars was 0.14 in. and the average displacement at the ultimate tensile load for uncoated reinforcing bars was 0.36 in. The experimental data from testing both the epoxy-coated and uncoated reinforcing bars is shown in Table 4.2. The failure mode for all of the epoxy-coated reinforcing bars was bond failure between the concrete and the chemical adhesive. The failure mode for all of the uncoated reinforcing bars was bond failure between the steel and the chemical adhesive. All the failure modes for reinforcing bars post-installed with Red Head A7+ are shown in Figure 4.2.

4.2.3 HILTI HIT RE-500 V3

The average ultimate tensile load for epoxy-coated reinforcing bars post-installed using Hilti HIT-RE 500 V3 was 33.00 kips. The average ultimate tensile load for uncoated reinforcing bars post-installed using Hilti HIT-RE 500 V3 was 33.76 kips (2% higher). The ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars to the uncoated reinforcing bars was 0.98. The average displacement at the ultimate tensile load for epoxy-coated reinforcing bars was 0.30 in. and the average displacement at the ultimate tensile load for uncoated reinforcing bars was 0.43 in. The experimental data from testing both the epoxy-coated and uncoated reinforcing bars is shown in Table 4.3. The failure mode for all of the epoxy-coated and uncoated reinforcing bars was steel rupture. The failure modes for reinforcing bars post-installed with Hilti HIT-RE 500 V3 are shown in Figure 4.3.

4.2.4 ATC ULTRABOND 365CC

The average ultimate tensile load for epoxy-coated reinforcing bars post-installed using ATC Ultrabond 365CC was 32.42 kips. The average ultimate tensile load for uncoated reinforcing bars post-installed using ATC Ultrabond 365CC was 31.00 kips (5% lower). The ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars to the uncoated reinforcing bars was 1.05. The average displacement at the ultimate tensile load for epoxy-coated reinforcing bars was 0.32 in. and the average displacement at the ultimate tensile load for uncoated reinforcing bars was 0.27 in. The experimental data from testing both the epoxy-coated and uncoated reinforcing bars is shown in Table 4.4. The failure mode for four of the epoxy-coated reinforcing bars was bond failure between the concrete and the chemical adhesive, and the failure mode of the remaining two epoxy-coated reinforcing bars was steel rupture. The failure mode for all of the uncoated reinforcing bars was bond failure between the steel and the chemical adhesive. All of the failure modes for reinforcing bars post-installed with ATC Ultrabond 365 CC are shown in Figure 4.4.

4.3 DISCUSSION

The ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars to the ultimate tensile pullout strength of the uncoated reinforcing bars was 0.94 for the Powers AC100+ Gold chemical adhesive. Similarly, the ratio was 0.97, 0.98, and 1.05 for the Red Head A7+, Hilti HIT-RE 500 V3, ATC Ultrabond 365CC products, respectively. The ratios of the ultimate tensile pullout strength of the epoxy-coated reinforcing bars to the ultimate tensile pullout strength of the uncoated reinforcing bars for all four of the chemical adhesives are summarized in Table 4.5. The standard deviation of each average ultimate tensile pullout strength was added or subtracted to the average ultimate tensile pullout strength of both the epoxy-coated or uncoated reinforcing bars to increase the ratio between the two reinforcing bar types. This methodology created more extreme ratios of the ultimate tensile pullout strength of the epoxy-coated reinforcing bars to the ultimate tensile pullout strength of the uncoated reinforcing bars. The ratios ranged from 0.88 to 0.99, as shown in Table 4.6. Furthermore, to encompass 99% of the normal distribution, the average ultimate tensile pullout strengths of both the epoxy-coated and uncoated reinforcing bars were reduced by three standard deviations as another means to investigate the ratio of average ultimate tensile pullout strength between the two types of bars. The minimum ratio was 0.89 as shown in Table 4.7, which corresponded to the minimum ratio of 0.88 from Table 4.6.

ATC Ultrabond 365CC was the only chemical adhesive that had a higher ultimate tensile pullout strength for the epoxy-coated reinforcing bars compared to the uncoated reinforcing bars. For all of the other chemical adhesives, the ultimate tensile pullout strength of the uncoated reinforcing bars was higher than that of the epoxy-coated reinforcing bars. Steel rupture was the only failure mode for bars installed with the Hilti HIT-RE 500 V3 adhesive, which means that the bond between the reinforcing bars, chemical adhesive, and concrete was stronger than the steel rupture strength. Therefore, the difference in bond strength between the epoxy-coated reinforcing bars and uncoated reinforcing bars installed with Hilti HIT-RE 500 V3 could not be compared.

A *t*-test was used to further analyze the laboratory experimental program results using statistics. The *t*-test is commonly used to determine whether the average of a data set significantly differs from the average of another data set (Ugoni and Walker, 1995). The *t*-test was used to determine if the average ultimate tensile pullout strength of the epoxy-coated reinforcing bars was significantly different than the average ultimate tensile pullout strength of the uncoated reinforcing bars. The *t*-test was performed as an unpaired *t*-test with a 5% significance level (α). The critical *t*-score was 2.228 based on the number of tensile pullout tests for epoxy-coated and uncoated reinforcing bars. The *t*-value for each chemical adhesive was calculated using the average ultimate tensile pullout strength and the standard deviation of the average ultimate tensile pullout strength. The *t*-value was compared to the critical *t*-score. If the *t*-value was greater than the critical *t*-score, then the average ultimate tensile pullout strengths were statistically different. If the *t*-value was less than the critical *t*-score, then the average ultimate tensile pullout strengths were not statistically different. The *t*-test analysis results are shown in Table 4.8.

Results from *t*-test analyses indicated that the average ultimate tensile pullout strength of the epoxy-coated reinforcing bars post-installed using Powers AC100+ Gold, Hilti HIT-RE 500 V3, and ATC Ultrabond 365CC were significantly different than the average ultimate tensile pullout strength of the uncoated reinforcing bars post-installed using the same respective chemical adhesive. This means that, statistically, the ultimate tensile pullout strength was not the same for both epoxy-coated and uncoated reinforcing bars post-installed using those three chemical adhesives. Results from the *t*-test analysis indicated that the average ultimate tensile pullout strength of the epoxy-coated reinforcing bars post-installed using Red Head A7+ were not significantly different than the average ultimate tensile pullout strength of the uncoated reinforcing bars post-installed using Red Head A7+. This means that, statistically, the ultimate tensile pullout strength was the same for both epoxy-coated and uncoated reinforcing bars post-installed using Red Head A7+.

The experimental ultimate tensile pullout strength of the epoxy-coated reinforcing bars post-installed using chemical adhesives were compared to the manufacturer bond strength values for uncoated reinforcing bars post-installed using chemical adhesives. The chemical adhesive manufacturers did not publish a value for just the bond strength, but rather published a value for the tensile capacity, which is the minimum of the steel strength, the concrete breakout strength, and the bond strength. Because of this, the bond strength values were calculated in accordance with the method from ACI 318 (2014) Section 17.4.5.1 (Equation 2.1 through Equation 2.6 in this document) using the τ_{uncr} listed in the ICC Evaluation Report for each chemical adhesive (ICC, 2018a; ICC, 2018b; ICC, 2017; ICC, 2016). The τ_{uncr} used for each chemical adhesive is shown in Table 4.9. It was found that the experimental ultimate tensile pullout strength of the epoxy-coated reinforcing bars was greater than the manufacturer bond strength values for all of the chemical adhesives. The lowest ratio of experimental ultimate tensile pullout strength of the epoxy-coated reinforcing bars divided by the manufacturer bond strength values for uncoated reinforcing bars was 1.63 (the largest was 3.16). A comparison of the experimental ultimate tensile pullout strength for epoxy-coated reinforcing bars to the manufacturer bond strength values for uncoated reinforcing bars, for all four of the chemical adhesives, is shown Table 4.10.

The experimental ultimate tensile pullout strength of the epoxy-coated and uncoated reinforcing bars post-installed using chemical adhesives were compared to the bond strength based on the MnDOT-specified uncracked bond stress (τ_{uncr}). MnDOT does not publish a value for the bond strength, but it was calculated using the MnDOT specified τ_{uncr} and the method from ACI 318 (2014) Section 17.4.5.1 (Equation 2.1 through Equation 2.6 in this document). The MnDOT-specified τ_{uncr} is shown in Table 4.9 and is 1,000 psi. The experimental ultimate tensile pullout strength for both epoxy-coated and uncoated reinforcing bars to the bond strength based on the MnDOT-specified τ_{uncr} for all four of the chemical adhesives is shown Table 4.10.

When examining the failure modes in this research, it was observed that bond failure occurred at the interface between the concrete and chemical adhesive for most of the epoxy-coated reinforcing bars. However, specimens ECU3, ECU5, and ECH1 through ECH6 did not follow this trend (approximately 33% of the epoxy-coated specimens). Bond failure occurred at the interface between the steel and the chemical adhesive for all of the specimens with uncoated reinforcing bar, except for those installed using the Hilti product (steel rupture failure). The failure modes in this research did not follow the failure mode results from Dickey (2011) and Meline et al. (2006). Two possible reasons for this could be that the chemical adhesives and embedment depths were different. The failure mode results from this research mean that the chemical adhesive adhered to the epoxy-coated reinforcing bars better than it adhered to the uncoated reinforcing bars. One possible explanation for this behavior is that the compounds in the chemical adhesives cause a chemical reaction with the epoxy-coating, which causes the two to bond better than the uncoated reinforcing bar. Despite the chemical adhesive adhering to the epoxy-coated reinforcing bars better than the uncoated reinforcing bars, it does not mean that the ultimate tensile pullout strength of the epoxy-coated reinforcing bars was greater than that of the uncoated reinforcing bars.

The average displacement at the ultimate tensile load for the epoxy-coated reinforcing bars post-installed using Powers AC100+ Gold, Red Head A7+, and Hilti HIT-RE 500 V3 was less than the average displacement at the ultimate tensile load for the uncoated reinforcing bars post-installed using the same respective chemical adhesive. The average displacement at the ultimate tensile load for the epoxy-coated reinforcing bars post-installed using ATC Ultrabond 365CC was more than the average displacement at the ultimate tensile load for the uncoated reinforcing bars post-installed using ATC Ultrabond 365CC. Plots of the applied tensile load vs. displacement for all of the tests are shown in Appendix B.

A *t*-test was also used to determine if the average displacement at the ultimate tensile load for the epoxy-coated reinforcing bars was significantly different than the ultimate tensile load for the uncoated reinforcing bars. Results from *t*-test analyses indicated that the average displacement at the ultimate tensile load for the epoxy-coated reinforcing bars post-installed using Powers AC100+ Gold, Red Head A7+, and Hilti HIT-RE 500 V3 were significantly different compared to the average displacement at the ultimate tensile load for the uncoated reinforcing bars post-installed using the same respective adhesive. This means that, statistically, the average displacement at the ultimate tensile load was not the same for both epoxy-coated and uncoated reinforcing bars post-installed using Powers AC100+ Gold, Red Head A7+, and Hilti HIT-RE 500 V3. Results from the *t*-test analysis indicated that the average displacement at the ultimate tensile load for the epoxy-coated reinforcing bars post-installed using ATC Ultrabond 365CC were not significantly different compared to the average displacement at the ultimate tensile load for the uncoated reinforcing bars post-installed using ATC Ultrabond 365CC. This means that, statistically, the average displacement at the ultimate tensile load was the same for both epoxy-coated and uncoated reinforcing bars post-installed using ATC Ultrabond 365CC. The *t*-test analysis results are shown in Table 4.11.

Table 4.1 Experimental results for both epoxy-coated and uncoated reinforcing bars post-installed with Powers AC100+ Gold adhesive

Specimen	Reinforcing bar coating	Ultimate load (kips)	Displacement at ultimate load (in.)	Failure mode
ECP1	Epoxy-coated	30.41	0.12	Concrete/adhesive bond
ECP2		31.35	0.11	
ECP3		30.43	0.15	
ECP4		29.40	0.10	
ECP5		27.76	0.08	
ECP6		29.49	0.08	
Average epoxy-coated		*29.81*	*0.11*	-
UCP1	Uncoated	30.82	0.30	Steel/adhesive bond
UCP2		30.68	0.19	
UCP3		32.40	0.29	
UCP4		31.48	0.32	
UCP5		31.50	0.21	
UCP6		32.42	0.40	
Average uncoated		*31.55*	*0.29*	-
Average epoxy-coated / Average uncoated =			*0.94*	

Table 4.2 Experimental results for both epoxy-coated and uncoated reinforcing bars post-installed with Red Head A7+ adhesive

Specimen	Reinforcing bar coating	Ultimate load (kips)	Displacement at ultimate load (in.)	Failure mode
ECR1	Epoxy-coated	30.37	0.15	Concrete/adhesive bond
ECR2		31.96	0.22	
ECR3		30.43	0.12	
ECR4		30.32	0.14	
ECR5		29.07	0.13	
ECR6		29.80	0.10	
Average epoxy-coated		*30.32*	*0.14*	-
UCR1	Uncoated	29.63	0.18	Steel/adhesive bond
UCR2		30.14	0.33	
UCR3		33.31	0.50	
UCR4		32.10	0.32	
UCR5		29.90	0.29	
UCR6		31.76	0.55	
Average uncoated		*31.27*	*0.36*	-
Average epoxy-coated / Average uncoated =			*0.97*	

Table 4.3 Experimental results for both epoxy-coated and uncoated reinforcing bars post-installed with Hilti HIT-RE 500 V3 adhesive

Specimen	Reinforcing bar coating	Ultimate load (kips)	Displacement at ultimate load (in.)	Failure mode
ECH1	Epoxy-coated	32.98	0.39	Steel rupture
ECH2		33.30	0.33	
ECH3		33.18	0.26	
ECH4		32.64	0.34	
ECH5		32.84	0.29	
ECH6		33.05	0.16	
Average epoxy-coated		*33.00*	*0.30*	-
UCH1	Uncoated	33.94	0.51	Steel rupture
UCH2		33.73	0.43	
UCH3		33.62	0.41	
UCH4		33.96	0.48	
UCH5		33.96	0.34	
UCH6		33.37	0.43	
Average uncoated		*33.76*	*0.43*	-
Average epoxy-coated / Average uncoated =			*0.98*	

Table 4.4 Experimental results for both epoxy-coated and uncoated reinforcing bars post-installed with ATC Ultrabond 365CC adhesive

Specimen	Reinforcing bar coating	Ultimate load (kips)	Displacement at ultimate load (in.)	Failure mode
ECU1	Epoxy-coated	33.13	0.40	Concrete/adhesive bond
ECU2		32.38	0.32	
ECU3		32.84	0.34	Steel rupture
ECU4		31.36	0.16	Concrete/adhesive bond
ECU5		32.68	0.46	Steel rupture
ECU6		32.14	0.26	Concrete/adhesive bond
Average epoxy-coated		*32.42*	*0.32*	-
UCU1	Uncoated	29.09	0.25	Steel/adhesive bond
UCU2		30.62	0.26	
UCU3		31.90	0.34	
UCU4		32.40	0.27	
UCU5		31.21	0.26	
UCU6		30.74	0.25	
Average uncoated		*31.00*	*0.27*	-
Average epoxy-coated / Average uncoated =			*1.05*	

Table 4.5 Summary of the average experimental results for both epoxy-coated and uncoated reinforcing bars

Adhesive type	Reinforcing bar coating	Average displacement at ultimate load (in.)	Average ultimate load (kips)	Ratio of epoxy-coated average ultimate load to uncoated average ultimate load
Powers AC100+ Gold	Epoxy-coated	0.11	29.81	0.94
	Uncoated	0.29	31.55	
Red Head A7+	Epoxy-coated	0.14	30.32	0.97
	Uncoated	0.36	31.27	
Hilti HIT-RE 500 V3*	Epoxy-coated	0.30	33.00	0.98
	Uncoated	0.43	33.76	
ATC Ultrabond 365CC	Epoxy-coated	0.32	32.42	1.05
	Uncoated	0.27	31.00	

*The failure mode of these bars was steel rupture and not bond.

Table 4.6 Ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars and the uncoated reinforcing bars including one standard deviation

Adhesive	Reinforcing bar coating	Average ultimate tensile pullout strength (kips)	Standard deviation of the average ultimate tensile pullout strengths (kips)	Average ultimate load including one standard deviation (kips)	Ratio of epoxy-coated to uncoated average ultimate tensile pullout strength including one standard deviation
Powers AC100+ Gold	Epoxy-coated	29.81	1.23	28.58	0.88
	Uncoated	31.55	0.75	32.30	
Red Head A7+	Epoxy-coated	30.32	0.95	29.37	0.90
	Uncoated	31.27	1.47	32.74	
Hilti HIT-RE 500 V3*	Epoxy-coated	33.00	0.24	32.76	0.96
	Uncoated	33.76	0.24	34.00	
ATC Ultrabond 365CC	Epoxy-coated	32.42	0.62	31.80	0.99
	Uncoated	31.00	1.16	32.16	

*The failure mode of these bars was steel rupture and not bond.

Table 4.7 Ratio of the ultimate tensile capacity of the epoxy-coated reinforcing bars and the uncoated reinforcing bars reduced by three standard deviations

Adhesive	Reinforcing bar coating	Average ultimate tensile pullout strength (kips)	Standard deviation of the average ultimate tensile pullout strengths (kips)	Average ultimate load reduced by three standard deviations (kips)	Ratio of epoxy-coated to uncoated average ultimate tensile pullout strength reduced by three standard deviations
Powers AC100+ Gold	Epoxy-coated	29.81	1.23	26.12	0.89
	Uncoated	31.55	0.75	29.30	
Red Head A7+	Epoxy-coated	30.32	0.95	27.47	1.02
	Uncoated	31.27	1.47	26.86	
Hilti HIT-RE 500 V3*	Epoxy-coated	33.00	0.24	32.28	0.98
	Uncoated	33.76	0.24	33.04	
ATC Ultrabond 365CC	Epoxy-coated	32.42	0.62	30.56	1.11
	Uncoated	31.00	1.16	27.52	

*The failure mode of these bars was steel rupture and not bond.

Table 4.8 Comparison of the ultimate tensile pullout strength of epoxy-coated reinforcing bars to uncoated reinforcing bars using *t*-test analyses

Adhesive	Reinforcing bar coating	Average ultimate tensile pullout strength (kips)	Standard deviation of the average ultimate tensile pullout strengths (kips)	Critical *t*-score	*t*-value	Statistically different (*t*-value > *t*-score)
Powers AC100+ Gold	Epoxy-coated	29.81	1.23	2.228	2.966	Yes
	Uncoated	31.55	0.75			
Red Head A7+	Epoxy-coated	30.32	0.95	2.228	1.139	No
	Uncoated	31.27	1.47			
Hilti HIT-RE 500 V3*	Epoxy-coated	33.00	0.24	2.228	5.572	Yes
	Uncoated	33.76	0.24			
ATC Ultrabond 365CC	Epoxy-coated	32.42	0.62	2.228	2.664	Yes
	Uncoated	31.00	1.16			

*The failure mode of these bars was steel rupture and not bond.

Table 4.9 Manufacturer published uncracked bond strength value (τ_{uncr}) from ICC Evaluation Reports for each chemical adhesive and MnDOT-specified τ_{uncr} (MnDOT, 2016)

Manufacturer or agency	Uncracked bond strength (τ_{uncr}), psi
Powers AC100+ Gold	1,117
Red Head A7+	1,900
Hilti HIT-RE 500 V3	1,720
ATC Ultrabond 365CC	1,044
MnDOT	1,000

Table 4.10 Experimental ultimate tensile pullout strength of the epoxy-coated reinforcing bars compared to the manufacturer bond strength value and MnDOT-specified bond strength value (MnDOT, 2016) for uncoated reinforcing bars

Adhesive type	Ultimate load (kips)		Ratio of average ultimate load for epoxy-coated and uncoated to manufacturer bond strength value
Powers AC100+ Gold	MnDOT bond strength*	9.82	
	Manufacturer bond strength*	10.97	-
	Tested epoxy-coated	29.81	2.72
	Tested uncoated	31.55	2.88
Red Head A7+	MnDOT bond strength*	9.82	
	Manufacturer bond strength*	18.65	-
	Tested epoxy-coated	30.32	1.63
	Tested uncoated	31.27	1.68
Hilti HIT-RE 500 V3	MnDOT bond strength*	9.82	
	Manufacturer bond strength*	16.89	-
	Tested epoxy-coated	33.00	1.95
	Tested uncoated	33.76	2.00
ATC Ultrabond 365CC	MnDOT bond strength*	9.82	
	Manufacturer bond strength*	10.25	-
	Tested epoxy-coated	32.42	3.16
	Tested uncoated	31.00	3.02

*Calculated using τ_{uncr} values from Table 4.9

Table 4.11 Comparison of the average displacement at the ultimate tensile load of epoxy-coated reinforcing bars to uncoated reinforcing bars using *t*-test analyses

Adhesive	Reinforcing bar coating	Average displacement at the ultimate tensile load (in.)	Standard deviation of the average displacement at the ultimate tensile load (in.)	Critical *t*-score	*t*-value	Statistically different (*t*-value > *t*-score)
Powers AC100+ Gold	Epoxy-coated	0.11	0.03	2.228	5.281	Yes
	Uncoated	0.29	0.08			
Red Head A7+	Epoxy-coated	0.14	0.04	2.228	3.708	Yes
	Uncoated	0.36	0.14			
Hilti HIT-RE 500 V3	Epoxy-coated	0.30	0.08	2.228	3.403	Yes
	Uncoated	0.43	0.06			
ATC Ultrabond 365CC	Epoxy-coated	0.32	0.11	2.228	1.141	No
	Uncoated	0.27	0.03			

Figure 4.1 Failure modes for epoxy-coated (concrete/adhesive bond) and uncoated (steel/adhesive bond) reinforcing bars post-installed with Powers AC100+ Gold adhesive

Figure 4.2 Failure modes for epoxy-coated (concrete/adhesive bond) and uncoated (steel/adhesive bond) reinforcing bars post-installed with Red Head A7+ adhesive

Figure 4.3 Steel rupture failure mode for epoxy-coated and uncoated reinforcing bars post-installed with Hilti HIT-RE 500 V3 adhesive

Figure 4.4 Failure modes for epoxy-coated (concrete/adhesive bond or steel rupture) and uncoated (steel/adhesive bond) reinforcing bars post-installed with ATC Ultrabond 365CC adhesive

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Results from *t*-test analyses indicated that, statistically, epoxy-coated reinforcing bars post-installed with Powers AC100+ Gold had a lower ultimate tensile pullout strength compared to uncoated reinforcing bars post-installed with the same respective chemical adhesive (94% lower). Results from the *t*-test analysis indicated that, statistically, epoxy-coated reinforcing bars post-installed with ATC Ultrabond 365CC had a higher ultimate tensile pullout strength compared to uncoated reinforcing bars post-installed with ATC Ultrabond 365CC (105%). Results from the *t*-test analysis indicated that, statistically, the ultimate tensile pullout strength was the same for both epoxy-coated and uncoated reinforcing bars post-installed using Red Head A7+. Results from the *t*-test analysis of reinforcing bars post-installed using Hilti HIT-RE 500 V3 were presented but were not used to analyze the differences in the ultimate tensile pullout strength between the epoxy-coated and uncoated bars because steel rupture was the only failure mode that occurred.

Due to the statistical inconsistency from the *t*-test, a broad conclusion could not be drawn in regard to how the epoxy coating affected the ultimate tensile pullout strength of the post-installed reinforcing bars considering all of the different chemical adhesives in one group. The difference in magnitude between the ultimate tensile pullout strength of the epoxy-coated and uncoated reinforcing bars varied between each of the four chemical adhesive manufacturers used in this research.

The lowest ratio of the experimental ultimate tensile pullout strength of the epoxy-coated reinforcing bars divided by the manufacturer published tensile strength values for uncoated reinforcing bars was 1.63 for Red Head A7+, and the highest ratio was 3.16 for ATC Ultrabond 365CC. It was concluded that the experimental ultimate tensile pullout strength of the epoxy-coated reinforcing bars was greater than the manufacturer published tensile strength values for all of the chemical adhesives. Possible reasons for this conclusion include the fact that chemical adhesive manufacturers are conservative when publishing values for the characteristic bond stress (τ_{uncr} and τ_{cr}) or the conditions in the laboratory at UMD were favorable. Having the experimental ultimate tensile pullout strength values of the epoxy-coated reinforcing bars be greater than the manufacturer published tensile strength values for uncoated reinforcing bars was verified in the literature by Meline et al. (2006).

MnDOT (2016) uses a specified uncracked bond stress (τ_{uncr}) of 1,000 psi when calculating the bond strength of a reinforcing bar post-installed using a chemical adhesive in accordance with ACI 318-14 Section 17.4.5.1 (Equation 2.1 through Equation 2.6 in this document). An uncracked bond stress (τ_{uncr}) value was back-calculated using the minimum experimental epoxy-coated ultimate tensile pullout strength (ECP5 had an ultimate tensile load of 27.76 kips), the reinforcing bar diameter (0.625 in.), and the effective embedment length (5 in.). The uncracked bond stress (τ_{uncr}) was back-calculated to be 2,827 psi, as shown in Equation 5.1. The back-calculated τ_{uncr} based on the minimum experimental epoxy-coated ultimate tensile pullout strength was more than the current τ_{uncr} that MnDOT uses by approximately 283%.

$$\tau_{uncr} = \frac{27.76\ \text{kips}}{1.0 * \pi * 0.625\ \text{in.} * 5\ \text{in.}} = 2{,}827\ \text{psi} \qquad 5.1$$

It was concluded that the average displacement at the ultimate tensile load for the epoxy-coated reinforcing bars was less than the average displacement at the ultimate tensile load for the uncoated reinforcing bars. Results from t-test analysis shown in Table 4.11 verified this conclusion. A possible explanation for this is that the compounds in the chemical adhesives cause a chemical reaction with the epoxy-coating, which causes the two to bond together. When this bond is broken, it may happen rapidly, causing only a small amount of displacement to occur. This conclusion means that signs of failure for epoxy-coated reinforcing bars post-installed using a chemical adhesive would be harder to observe than those of uncoated reinforcing bars. In an extreme event load case, where loading occurs quickly (i.e., a post-installed bridge barrier or a post-installed bridge pier crash strut), this phenomenon would not be much of a concern because other warning signs would be evident. In other non-extreme event applications, having less deflection could be an issue because it would be harder to detect signs of failure.

5.2 RECOMMENDATIONS

Recommendations are made based on the results from the laboratory experimental program. The first recommendation is to include a modification factor when calculating bond strength for an epoxy-coated reinforcing bar post-installed using chemical adhesives ($\psi_{e,Na}$). Application of this modification factor would be similar to the method used in the development length equation from ACI 318 (2014) for cast-in-place reinforcement (Equation 2.7 in this document). For epoxy-coated reinforcing bars post-installed using chemical adhesives, it is recommended that $\psi_{e,Na}$ = 0.9 and be applied to Equation 2.1. The bar coating modification factor name was chosen as $\psi_{e,Na}$ because it follows the naming convention used by ACI 318 (2014) Chapter 17. The subscript "*e*" denotes an epoxy-coating. The bar coating modification factor value of 0.9 was chosen because it encapsulates the lowest ratio of the ultimate tensile pullout strength of the epoxy-coated reinforcing bars to the ultimate tensile pullout strength of the uncoated reinforcing bars from the laboratory experimental program, while still providing a built-in factor of safety. Two statistical methods were used to recommend the bar coating modification factor value of 0.9. In the first method, one standard deviation was applied to increase the ratio between the epoxy-coated and uncoated reinforcing bars with results shown in Table 4.6. The second method encompassed 99% of the normal distribution for the average ultimate tensile pullout strength of both the epoxy-coated and uncoated reinforcing bars by reducing the average for both bars by three standard deviations with results shown in Table 4.7. The minimum ratio from the first method was 0.88 and the minimum ratio from the second method was 0.89. Both minimum ratios are nearly bounded by the recommended value of 0.9. All of the other ratios from the two statistical methods are bounded by the 0.9 value. The proposed bond strength equation with a bar coating modification factor would take the form below.

$$N_a = \frac{A_{Na}}{A_{Nao}} \psi_{ed,Na} \, \psi_{cp,Na} \psi_{e,Na} \, N_{ba} \tag{5.2}$$

The second recommendation is that MnDOT raise the specified uncracked bond stress (τ_{uncr}) of 1,000 psi or use the manufacturer published values for τ_{uncr}. The back-calculated τ_{uncr} based on the minimum experimental epoxy-coated ultimate tensile pullout strength was approximately 283% more than the current τ_{uncr} specified by MnDOT. If MnDOT were to use the manufacturer published values for τ_{uncr}, its designs would better follow the requirements of Section 5.13.2.3 from the AASHTO LRFD Bridge Design Specifications (2017) and Section 17.4.5.2 from ACI 318 (2014). Alternatively, raising the current τ_{uncr} would provide more capacity during design calculations but would still not explicitly follow the requirements of the AASHTO LRFD Bridge Design Specifications (2017) and ACI 318 (2014).

REFERENCES

ACI Committee 318. (2011). *Building code requirements for structural concrete (ACI 318-11) and commentary.* American Concrete Institute, Farmington Hills, MI: ACI.

ACI Committee 318. (2014). *Building code requirements for structural concrete (ACI 318-14) and commentary.* American Concrete Institute, Farmington Hills, MI: ACI.

ACI Committee 224. (2001). *Control of Cracking in Concrete Structures (ACI 224R-01)*. American Concrete Institute, Farmington Hills, MI: ACI.

ACI Committee 355. (2011). *Qualification of post-installed adhesive anchors in concrete (ACI 355.4-11)*. American Concrete Institute, Farmington Hills, MI: ACI.

American Institute of Steel Construction (AISC). (2011). *Steel Construction Manual 14th Ed*. Chicago, IL: AISC.

American Association of State and Highway Transportation Officials (AASHTO). (2017). *AASHTO LRFD bridge design specifications 8th edition*. Washington, DC: AASHTO.

Anderson, N., S. and Meinheit, D., F. (2014). Post-installed anchors the present state of the industry. *Structure Magazine,* 11 (8), 32–33.

Arkansas Department of Transportation. *Qualified Products List – Division 507 – Resin Anchor Systems.* Little Rock, AR: ArDOT.

ASTM International. (1996). ASTM *E488-96 Standard Test Methods for Strength of Anchors in Concrete Elements*. West Conshohocken, PA: ASTM International.

ASTM International. (2015). ASTM *E488-15 Standard Test Methods for Strength of Anchors in Concrete Elements*. West Conshohocken, PA: ASTM International.

ASTM International. (2016). *ASTM A615-16 Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement.* West Conshohocken, PA: ASTM International.

ASTM International. (2017). *ASTM A775-17 Standard Specification for Epoxy-Coated Steel Reinforcing Bars.* West Conshohocken, PA: ASTM International.

California Department of Transportation. (2012). *Bridge Design Aid 5-81*. Sacramento, CA: Caltrans.

California Department of Transportation. (2015). *Standard Specifications.* Sacramento, CA: Caltrans.

Canadian Standards Association (CSA). (2014). *Canadian highway bridge design code*. Mississauga, Ontario: CSA.

Canadian Standards Association (CSA). (2014). *Design of concrete structures (A23.3-14)*. Mississauga, Ontario: CSA.

Carlson, R., Houghton, D. and Polivka, M. (1997). Causes and Control of Cracking in Unreinforced Mass Concrete. *ACI Journal*, 76 (7), 821-838.

Choi, O., Darwin, D. and McCabe, S. (1990). *Bond Strength of Epoxy-Coated Reinforcement to Concrete*. (SM Report No. 25). Washington, DC: National Science Foundation.

Cook, R., A. Kunz, J. Fuchs, W. and Konz, R., C. (1998). Behavior and design of single adhesive anchors under tensile load in uncracked concrete. *ACI Structural Journal*, 95 (1), 9-26.

Dickey, B. (2011). Development of a design procedure for concrete traffic barrier attachments to bridge decks utilizing epoxy concrete anchors. Master's thesis, University of Nebraska, Lincoln, NE.

Ehrlich, A. (October 19, 2017). *MEMO TO DESIGNERS (2017-02): Post-Installed Anchorages for Reinforcing Bars* [Memorandum]. Oakdale, MN: MnDOT.

Eligehausen, R. Mahrenholtz, C. Akguzel, U. and Pampanin, S. (2012). Bridging the gap between design provisions for connections using anchorage or strut-and-tie models. *ACI Special Publication 296*, 79–94, American Concrete Institute, Farmington Hills, MI: ACI.

European Committee for Concrete (CEB) and International Federation for Prestressing (FIP). (2010). *Model Code for Concrete Structures.* Lausanne, Switzerland: CEB and FIP.

Hartt, W., Powers, R., Lysogorski, D., Liroux, V. and Virmani, P. (2007). *Corrosion Resistant Alloys for Reinforced Concrete*. (Report No. FHWA-HRT-07-039). Federal Highway Administration, Springfield, VA: FHWA.

Hörmann-gast, A. and Silva, J. (2012). Revisions to ICC-ES acceptance criteria for post-installed adhesive anchors, AC308. *ICC Evaluation Service*, 308 (6), 5–7.

International Code Council (ICC). (2010). Acceptance Criteria for Adhesive Anchors in Masonry Elements (AC 58). Washington, DC: ICC.

ICC. (2013). Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements (AC 308). Washington, DC: ICC.

ICC. (2016). *ESR-3903 ITW Red Head A7+ Adhesive Anchoring Systems for Cracked and Uncracked Concrete*. Washington, DC: ICC.

ICC. (2017). *ESR-3814 Hilti HIT-RE 500 V3 Adhesive Anchors and Post-Installed Reinforcing Bar Connections in Cracked and Uncracked Concrete*. Washington, DC: ICC.

ICC. (2018). *ESR-3770 Adhesive Technology Corporation Ultrabond 365CC Adhesive Anchoring System for Cracked and Uncracked Concrete*. Washington, DC: ICC.

ICC. (2018). *ESR-2582 AC100+ Gold Adhesive Anchor System in Cracked and Uncracked Concrete*. Washington, DC: ICC.

Iowa Department of Transportation. (2016). *IM 491.11 Polymer Grouts and Adhesive Anchors.* Ames, IA. https://iowadot.gov/erl/current/IM/content/491.11.htm

Johnston, D. W. and Zia, P. (1982). *Bond characteristics of epoxy-coated reinforcing bars*. (Report No. FHWA/NC/82-002). North Carolina Department of Transportation, Raleigh, NC: NCDOT.

Lutz, L., A. Gergely, P. and Winter, G. (1966). The Mechanics of bond and slip of deformed reinforcing bars in concrete. *ACI Journal*, 64 (11), 711-721.

Mathey, R., G. and Clifton, J., R. (1976). Bond of coated reinforcing bars in concrete. *Journal of the Structural Division ASCE*, 102, 215-228.

Meline, R. J., Gallaher, M. and Duane, J. (2006). *Seismic, creep, and tensile testing of various epoxy bonded rebar products in hardened concrete*. (Report No. CA06-0247). California Department of Transportation, Sacramento, CA: Caltrans.

Michigan Department of Transportation. (2017). *Bridge Design Manual*. Detroit, MI: MDOT.

Michigan Department of Transportation. (2018). *Qualified Products List*. Detroit, MI: MDOT. https://www.michigan.gov/documents/MDOT-Material_Source_Guide_Qualified_Products_84764_7.pdf

MnDOT. (2016). *Approved Concrete Adhesive Anchorage Program.* Oakdale, MN: MnDOT.

MnDOT. (2017). *LRFD Bridge Design Manual*. Oakdale, MN: MnDOT.

Treece, R., A. and Jirsa, J., O. (1989). Bond strength of epoxy-coated reinforcing bars. *ACI Materials Journal*, 86 (2), 167-174.

Ugoni, A. and Walker, B. (1995). The *t*-test an Introduction. *COMSIG Review, 4*(2), 37-40.

Zamora, N., Cook, R., Konz, R. and Consolazio, G., (2003). Behavior and Design of Single, Headed and Unheaded, Grouted Anchors under Tensile Load. *ACI Structural Journal*, 100 (2), 222-230.

APPENDIX A: DESIGN FLOWCHART AND CALCULATIONS FOR A POST-INSTALLED BRIDGE BARRIER AND A POST-INSTALLED BRIDGE PIER CRASH STRUT

Note

The purpose of this flowchart is to provide guidance on the design of rebar post-installed with a chemical adhesive. The flowchart does not examine all of the steel and concrete failure modes for post-installed anchors, but rather just the failure modes applicable to rebar post-installed with a chemical adhesive.

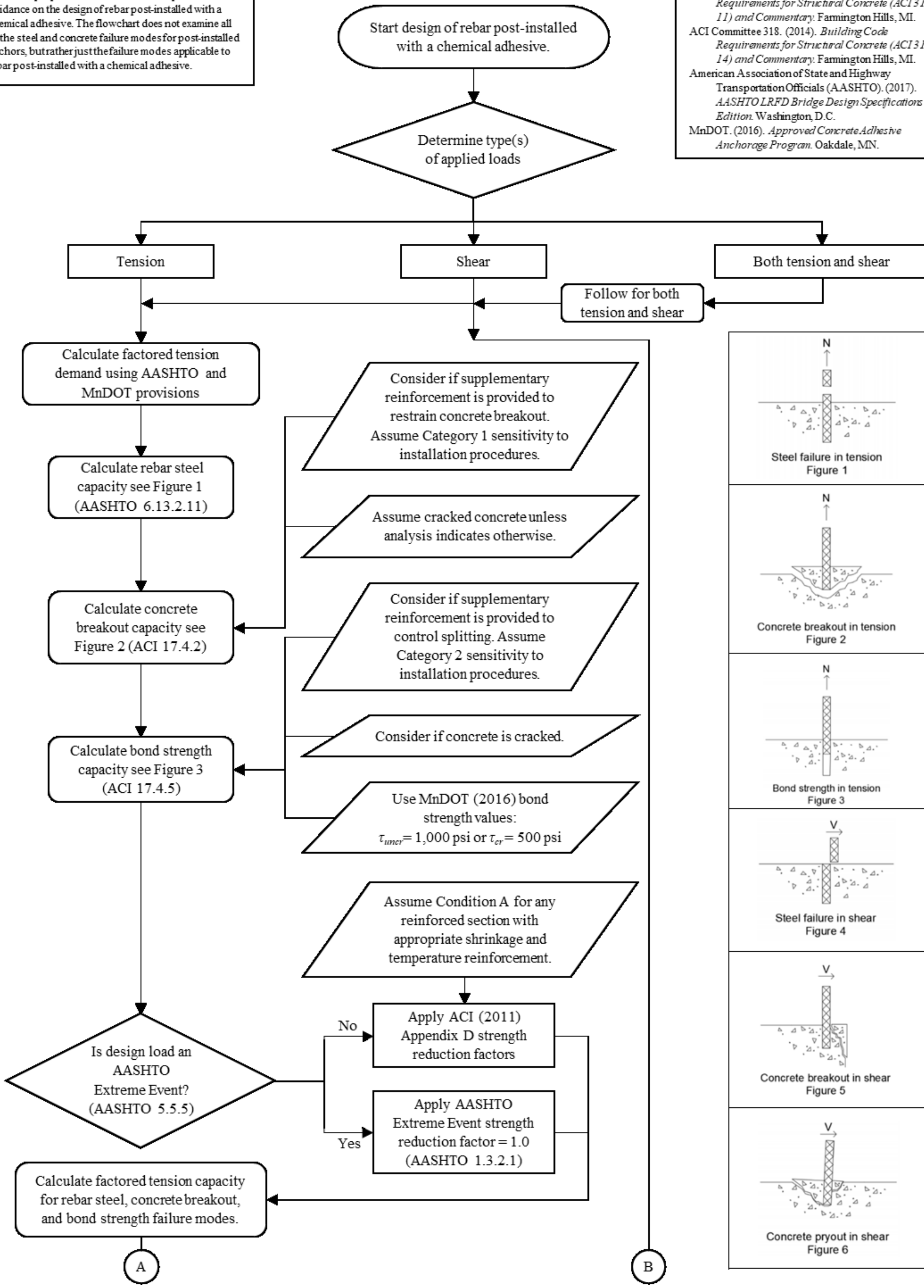
References

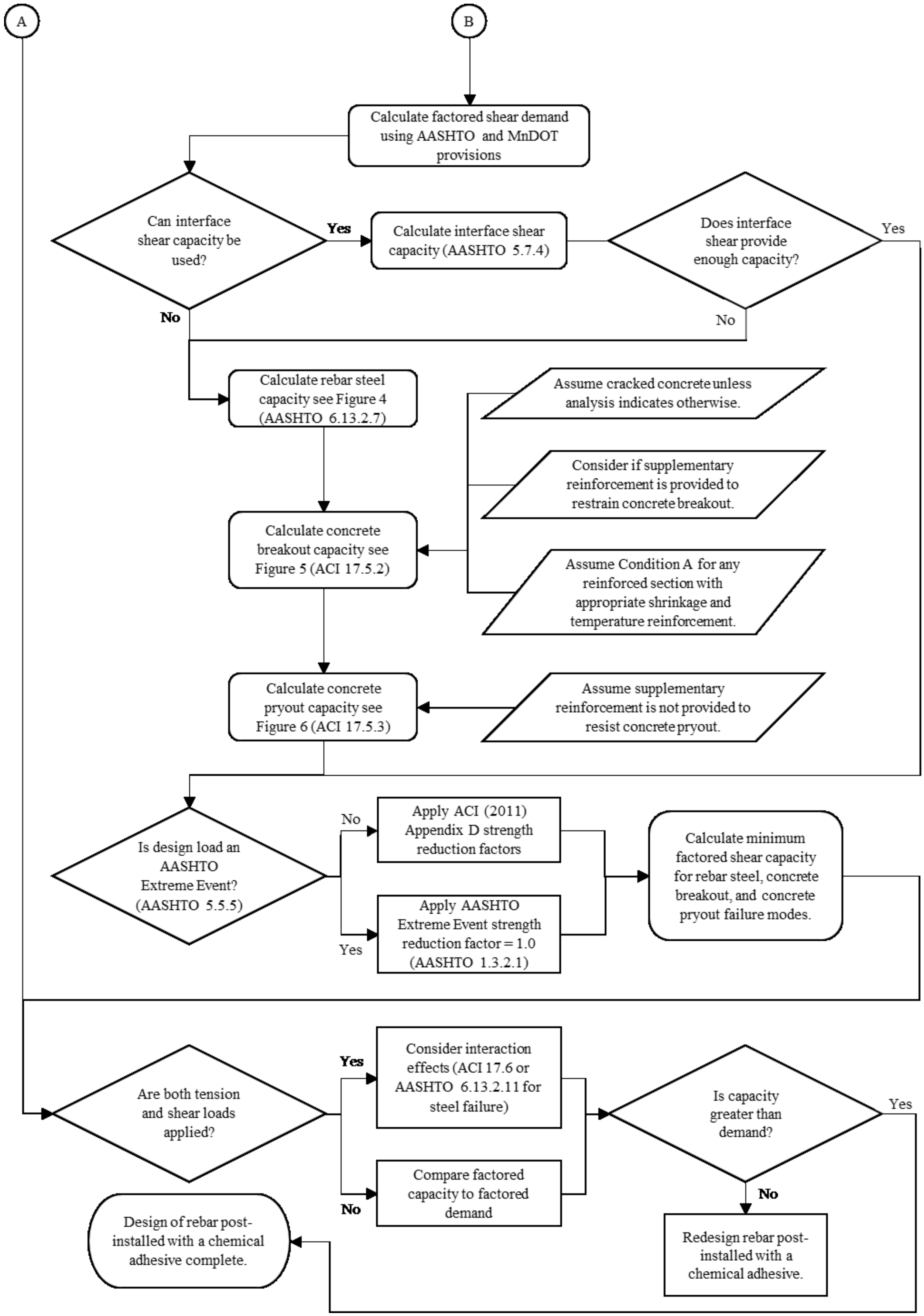
ACI Committee 318. (2011). *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary.* Farmington Hills, MI.

ACI Committee 318. (2014). *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary.* Farmington Hills, MI.

American Association of State and Highway Transportation Officials (AASHTO). (2017). *AASHTO LRFD Bridge Design Specifications 8th Edition.* Washington, D.C.

MnDOT. (2016). *Approved Concrete Adhesive Anchorage Program.* Oakdale, MN.




A
B
Calculate factored shear demand using AASHTO and MnDOT provisions
Can interface shear capacity be used?
Yes
No
Calculate interface shear capacity (AASHTO 5.7.4)
Does interface shear provide enough capacity?
Yes
No
Calculate rebar steel capacity see Figure 4 (AASHTO 6.13.2.7)
Assume cracked concrete unless analysis indicates otherwise.
Consider if supplementary reinforcement is provided to restrain concrete breakout.
Calculate concrete breakout capacity see Figure 5 (ACI 17.5.2)
Assume Condition A for any reinforced section with appropriate shrinkage and temperature reinforcement.
Calculate concrete pryout capacity see Figure 6 (ACI 17.5.3)
Assume supplementary reinforcement is not provided to resist concrete pryout.
Is design load an AASHTO Extreme Event? (AASHTO 5.5.5)
No
Yes
Apply ACI (2011) Appendix D strength reduction factors
Apply AASHTO Extreme Event strength reduction factor = 1.0 (AASHTO 1.3.2.1)
Calculate minimum factored shear capacity for rebar steel, concrete breakout, and concrete pryout failure modes.
Are both tension and shear loads applied?
Yes
No
Consider interaction effects (ACI 17.6 or AASHTO 6.13.2.11 for steel failure)
Compare factored capacity to factored demand
Is capacity greater than demand?
Yes
No
Redesign rebar post-installed with a chemical adhesive.
Design of rebar post-installed with a chemical adhesive complete.

Type S Barrier Retrofit Design Example

In this example, a retrofit barrier is attached to an existing concrete bridge deck using rebar post-installed with a chemical adhesive. The barrier is a MnDOT standard 36 in. Type S, TL-4. The cross-section of the barrier and the rebar sizes and types are illustrated in Figure 1 (based on the Bridge Details Part II FIG. 5-397.138(A) Section B-B). The geometry of the existing concrete bridge deck is shown in Figure 2. In this example, both the cast-in-place longitudinal rebar and the post-installed vertical rebar is checked for tensile capacity, for both an interior and end region section of barrier. The barrier design follows the method presented in the 8th Edition of AASHTO in Section A13.3.1. The post-installed rebar design follows Ch. 17 of ACI 318-14, except for steel strength, which follows AASHTO Section 6.13.2.11. Shear design followed AASHTO Section 5.7.4, and interface shear was used to check the capacity.

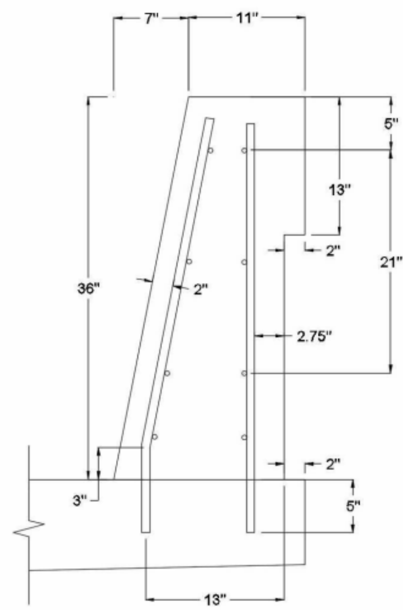


Figure 1. Cross section of retrofit barrier

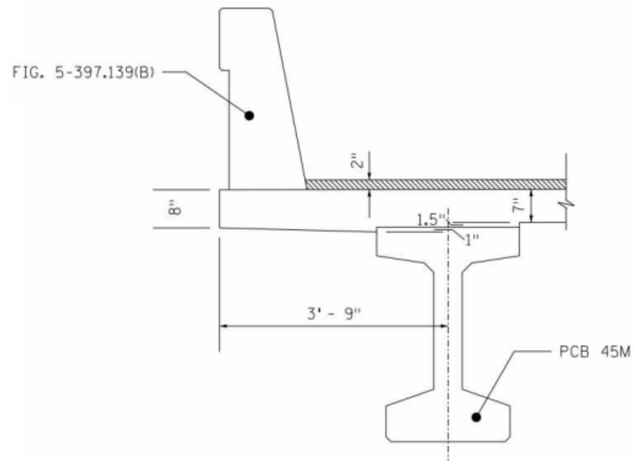


Figure 2. Geometry of existing concrete bridge deck

(13.7.3.2) ***Barrier load demand:***

(NCHRP, 1993) MnDOT Type S barriers satisfy the requirements for both MASH TL-4 and NCHRP 350 TL-4. However, the retrofit barrier in this example is designed for NCHRP 350 TL-4. The design forces and dimensional limits for a NCHRP 350 TL-4 barrier are presented in AASHTO Table A13.2-1.

(Table 13.7.2-1)

Design Forces and Designations	TL-4
F_t Transverse (kip)	54.0
F_L Longitudinal (kip)	18.0
F_v Vertical (kip)	18.0
L_t and L_L (ft)	3.5
L_v (ft)	18.0
H_e Minimum Height of Horizontal Loads (in.)	32.0
Minimum H Height of Rail (in.)	32.0

Moment capacity of barrier:

(A13.3.1) The design is based on yield-line analysis methods presented in AASHTO and has three capacity variables:

- M_c – the flexural resistance of the railing about a horizontal axis
- M_w – the flexural resistance of the railing about its vertical axis
- M_b – the flexural resistance of the cap beam (not present in this example)

The barrier capacity is different for interior regions compared to exterior regions. The critical barrier length (L_c) is different for each situation. The yield-line for an interior barrier region is shown in Figure 3 and the yield-line for an exterior barrier region is shown in Figure 4.

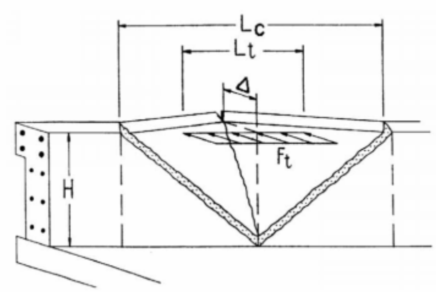


Figure 3. Yield-line for interior region (AASHTO CA13.3.1-1)

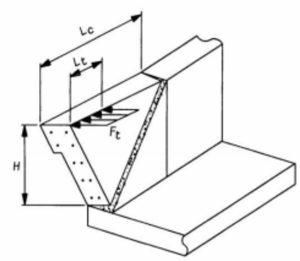


Figure 4. Yield-line for exterior region (AASHTO CA13.3.1-1)

Barrier interior region:

(A13.3.1) Determine the flexural resistance of a cantilevered railing about an axis parallel to the longitudinal axis of the bridge for an interior barrier region (M_c).

The elevation of the interior region retrofit barrier with reinforcement and post-installed rebar is shown in Figure 5.

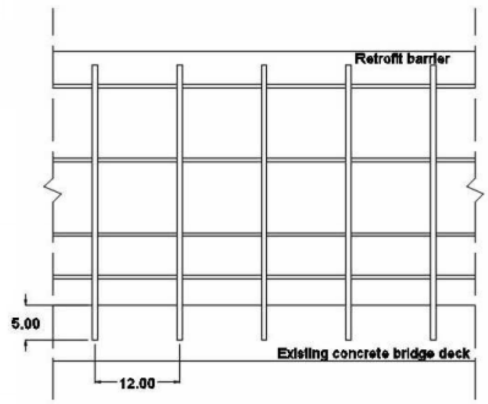


Figure 5. Elevation view of retrofit barrier interior region (units of in.)

Design assumptions:

For this example #6 rebar is post-installed

Rebar properties: $d_{a\#6} := 0.75 \cdot in$ $A_{s\#6} := 0.44\ in^2$

$f_y := 60\ ksi$ $f_u := 90\ ksi$

Effective embedment: $h_{ef} := 5 \cdot in$

The effective embedment depth of 5 in. was chosen to avoid interferance with the bottom layer of deck rebar.

(ACI 17.2.7) Concrete compressive strength: $f'_c := 4000\ psi$

Note that ACI 318 equations require f'_c to be given in psi.

Unit weight of concrete: $w_c := 150\ pcf$

(ACI 19.2.4.2) Lightweight concrete modification factor: $\lambda_a := 1.0$ For normal weight concrete

Post-installed rebar spacing: $rebar\_spa := 12\ in$

Area of post-installed rebar per foot: $A_s := \dfrac{A_{s\#6}}{rebar\_spa} = 0.44\ \dfrac{in^2}{ft}$

Tensile capacity of rebar post-installed with a chemical adhesive needs to be calculated in accordance with ACI 318-14 to calculate M_c. Three tensile capacities are checked, which include steel strength, concrete breakout strength, and bond strength. Illustrations of the three tensile capacities are shown in Figure 6. ACI-318 Table 17.3.1.1 also presents pullout strength in tension (ACI 17.4.3) and concrete side-face blowout strength in tension (ACI 17.4.4) as tensile capacities to be checked, however these tensile capacities are not applicable to post-installed rebar.

Figure 6. Tensile capacity failure modes

Only tension is applied to the post-installed rebar because it is assumed that interface shear between the retrofit barrier and the existing concrete bridge deck provide the required shear capacity (anchorage shear is not checked).

(6.13.2.10.2) Steel strength in tension:

$$N_{saPerFt} := 0.76 \cdot A_s \cdot f_u = 30.1 \ \frac{kip}{ft}$$

(1.3.2.1) Extreme event phi factor: $\phi := 1.0$

$$\phi N_{saPerFt} := \phi \cdot N_{saPerFt} = 30.1 \ \frac{kip}{ft}$$

(ACI 17.4.2) Concrete breakout strength due to tension on anchor:

(ACI 17.4.2.1c) Projected influence area not considering edge distances:

$$A_{Nco} := 9 \ h_{ef}^{\ 2} = 225 \ in^2$$

(ACI Fig R17.4.2.1) Projected influence area considering edge distances and group effects:

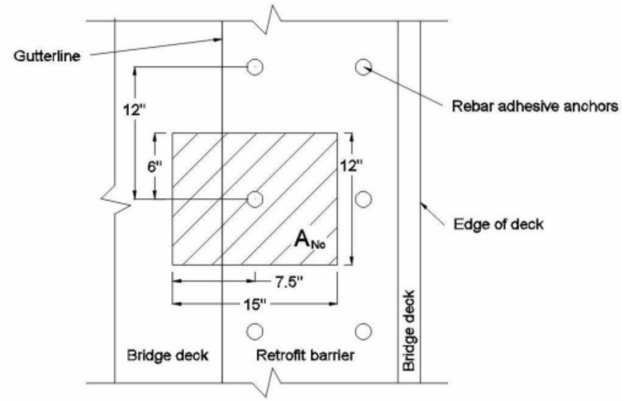


Figure 7. Projected influence area (A_{Nc})

$rebar\_spa = 12$ ***in***

$1.5 \cdot h_{ef} = 7.5$ ***in*** > $\dfrac{rebar\_spa}{2} = 6$ ***in***

Post-installed rebar is treated as a group because $1.5\,h_{ef}$ is greater than half the rebar spacing

$$A_{Nc} := \left(2 \cdot 1.5 \cdot h_{ef}\right) \cdot \left(2 \cdot \left(\frac{rebar\_spa}{2}\right)\right) = 180\ \boldsymbol{in}^2$$

(ACI 17.4.2.2a) Basic concrete breakout strength:

$h_{ef} = 5$ ***in*** < 11 in., so: $k_c := 17$

$$N_b := k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot \left(h_{ef}\right)^{1.5} \cdot \sqrt{\boldsymbol{lbf} \div \boldsymbol{in}} = 12.0\ \boldsymbol{kip}$$

(ACI 17.4.2.4) Modification factor for eccentric loading:

Assume no load is applied eccentrically

$\psi_{ec.N} := 1.0$

(ACI 17.4.2.5) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from the center of the rebar to the edge of the concrete bridge deck as shown in Figure 8.

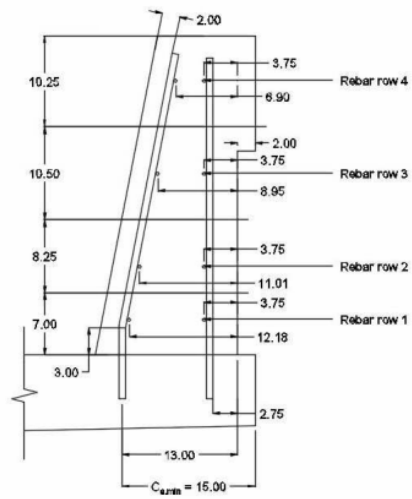


Figure 8. Rebar geometry required for calculation
of capacity (units of in.)

Post-installed rebar needs 2" clear cover on the traffic face of the barrier and the rebar needs to be straight for 3" before being bent, see Figure 8 for $c_{a.min}$.

$c_{a.min} := 15\ \textbf{in}$

$c_{a.min} = 15\ \textbf{in}\ >\ 1.5 \cdot h_{ef} = 7.5\ \textbf{in}$ so, $\psi_{ed.N} := 1.0$

(ACI 17.4.2.6) Modification factor for cracking:

Assume that the deck concrete is cracked

$\psi_{c.N} := 1.0$

(ACI 17.4.2.7) Modification factor for splitting:

Modification factor is not considered because it was assumed that the deck concrete is cracked.

(ACI 17.4.2.1b) Concrete breakout strength in tension for a group of post-installed rebar:

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot N_b = 9.6\ \textbf{kip}$$

Concrete breakout strength resistance reduction factor:

(1.3.2.1) Extreme event phi factor: $\phi_{cb} := 1.0$

Unfactored concrete breakout strength in tension per foot:

$$N_{cb.PerFt} := N_{cb} \cdot \frac{1}{rebar\_spa} = 9.6\ \frac{kip}{ft}$$

Factored concrete breakout strength in tension per foot:

$$\phi N_{cb.PerFt} := \phi_{cb} \cdot N_{cb.PerFt} = 9.6\ \frac{kip}{ft}$$

(ACI 17.4.5) Bond strength:

Although the deck was considered cracked for concrete breakout, it is expected that there will be few, if any, cracks that run through the adhesive around the anchor. Therefore, assume that the concrete bridge deck will be uncracked for the bond strength check. MnDOT (2016) specifies a bond strength value for uncracked concrete.

(MnDOT, 2016) $\tau_{uncr} := 1000 \cdot psi$

(ACI 17.4.5.1d) Projected distance to develop bond strength:

$$c_{Na} := 10 \cdot d_{a\#6} \cdot \sqrt{\frac{\tau_{uncr}}{1100 \cdot psi}} = 7.15\ in$$

(ACI 17.4.5.1c and ACI Fig R17.4.5.1) Projected influence area not considering edge distances:

$$A_{Nao} := \left(2 \cdot c_{Na}\right)^2 = 204.5\ in^2$$

(ACI Fig R17.4.5.1) Projected influence area considering edge distances and group effects:

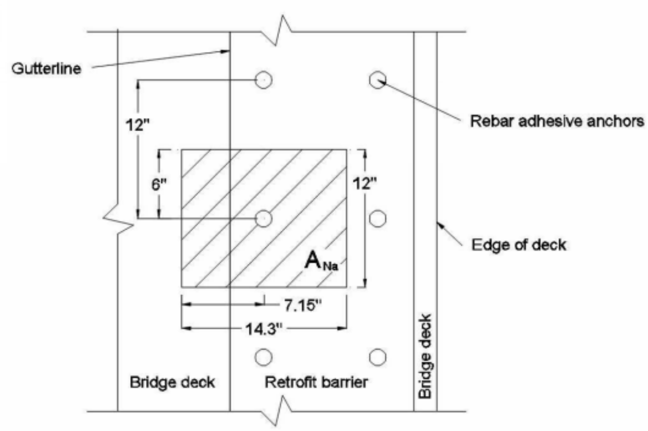


Figure 9. Projected influence area (A_{Na})

Because c_{Na} is greater than half of the post-installed rebar spacing, the rebar is treated as an anchor group for bond strength

$$c_{Na}=7.15\ \mathbf{in} \qquad > \qquad \frac{rebar\_spa}{2}=6\ \mathbf{in}$$

$$A_{Na}:=\left(2 \cdot c_{Na}\right) \cdot rebar\_spa=171.6\ \mathbf{in}^2$$

(ACI 17.4.5.2) Basic bond strength:

Concrete is assumed to be uncracked

$$N_{ba}:=\lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_{a\#6} \cdot h_{ef}=11.8\ \mathbf{kip}$$

(ACI 17.4.5.4) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from the center of the rebar to the edge of the concrete.

Post-installed rebar needs 2" clear cover on the traffic face of the barrier and the rebar needs to be straight for 3" before being bent, see Figure 8 for $c_{a.min}$.

$$c_{a.min}:=15\ \mathbf{in}$$

$$c_{a.min}=15\ \mathbf{in} \ > \ c_{Na}=7.15\ \mathbf{in} \qquad \text{so,} \quad \psi_{ed.Na}:=1.0$$

(ACI 17.4.5.5) Modification factor for splitting:

Supplementary reinforcement is provided in the bridge deck

$$\psi_{cp.Na}:=1.0$$

(ACI 17.4.5.1a) Bond strength in tension per post-installed rebar:

$$N_a:=\frac{A_{Na}}{A_{Nao}} \cdot \psi_{ed.Na} \cdot \psi_{cp.Na} \cdot N_{ba}=9.9\ \mathbf{kip}$$

Bond strength resistance reduction factor

(1.3.2.1) Extreme event phi factor: $\phi_{ba}:=1.0$

Unfactored bond strength in tension per foot:

$$N_{a.PerFt}:=N_a \cdot \frac{1}{rebar\_spa}=9.9\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

Factored bond strength in tension per foot:

$$\phi N_{a.PerFt} := \phi_{ba} \cdot N_{a.PerFt} = 9.9\ \frac{kip}{ft}$$

Barrier resistance:

(A13.3.1) To calculate M_c, the minimum tensile capacity of rebar post-installed with a chemical adhesive was calculated in accordance with ACI 318-14 and AASHTO. The moment capacity is calculated using the internal lever arm and Whitney's equivalent rectangular stress block.

Depth of Whitney's equivalent rectangular stress block per foot calculated using the minimum unfactored tensile capacity from the post-installed rebar design instead of the rebar yield strength:

$$F := min\left(N_{cb.PerFt}, N_{a.PerFt}, N_{saPerFt}\right) = 9.6\ \frac{kip}{ft}$$

$$a := \frac{F}{0.85 \cdot f'_c} = 0.236\ in/ft$$

The internal lever arm (d) is shown in Figure 8.

$$d := 13\ in$$

The internal lever arm (d) at the bottom of the barrier height was selected because that is the critical location of the yield-line capacity for post-installed rebar. This value of d matches the assumption made for calculating a single Whitney's equivalent rectangular stress block depth (a) at the bottom of the barrier. For CIP barriers, the internal lever arm (d) is typically averaged over the barrier height.

$$M_c := min\left(\phi N_{cb.PerFt}, \phi N_{a.PerFt}, \phi N_{saPerFt}\right) \cdot \left(d - \frac{a}{2}\right)$$

$$M_c = 10.3\ \frac{kip \cdot ft}{ft}$$

Determine the flexural resistance of the railing about its vertical axis for an interior region (M_w).

$$M_w = M_{wfront} + M_{wback}$$

Area of longitudinal rebar: $A_{s\#4} := 0.2\ in^2$

Moments calculated using traffic side rebar as shown in Figure 8

$d_{1front} := 12.18\ \boldsymbol{in}$ $\quad b_{1front} := 7.00\ \boldsymbol{in}$
$d_{2front} := 11.01\ \boldsymbol{in}$ $\quad b_{2front} := 8.25\ \boldsymbol{in}$ (1=bottom rebar row,
$d_{3front} := 8.95\ \boldsymbol{in}$ $\quad b_{3front} := 10.50\ \boldsymbol{in}$ 4=top rebar row)
$d_{4front} := 6.90\ \boldsymbol{in}$ $\quad b_{4front} := 10.25\ \boldsymbol{in}$

$$M_{1front} := A_{s\#4} \cdot f_y \cdot \left(d_{1front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{1front}} \right) = 11.9\ \boldsymbol{kip \cdot ft}$$

$$M_{2front} := A_{s\#4} \cdot f_y \cdot \left(d_{2front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{2front}} \right) = 10.8\ \boldsymbol{kip \cdot ft}$$

$$M_{3front} := A_{s\#4} \cdot f_y \cdot \left(d_{3front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{3front}} \right) = 8.8\ \boldsymbol{kip \cdot ft}$$

$$M_{4front} := A_{s\#4} \cdot f_y \cdot \left(d_{4front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{4front}} \right) = 6.7\ \boldsymbol{kip \cdot ft}$$

Moments calculated using non-traffic side rebar as shown in Figure 8. Although the top bar has an additional 2" from coping, it is not considered for the moment capacity.

$d_{1back} := 3.75\ \boldsymbol{in}$ $\quad b_{1back} := 7.00\ \boldsymbol{in}$
$d_{2back} := 3.75\ \boldsymbol{in}$ $\quad b_{2back} := 8.25\ \boldsymbol{in}$ (1=bottom rebar row,
$d_{3back} := 3.75\ \boldsymbol{in}$ $\quad b_{3back} := 10.50\ \boldsymbol{in}$ 4=top rebar row)
$d_{4back} := 3.75\ \boldsymbol{in}$ $\quad b_{4back} := 10.25\ \boldsymbol{in}$

$$M_{1back} := A_{s\#4} \cdot f_y \cdot \left(d_{1back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{1back}} \right) = 3.5\ \boldsymbol{kip \cdot ft}$$

$$M_{2back} := A_{s\#4} \cdot f_y \cdot \left(d_{2back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{2back}} \right) = 3.5\ \boldsymbol{kip \cdot ft}$$

$$M_{3back} := A_{s\#4} \cdot f_y \cdot \left(d_{3back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{3back}} \right) = 3.6\ \boldsymbol{kip \cdot ft}$$

$$M_{4back} := A_{s\#4} \cdot f_y \cdot \left(d_{4back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{4back}} \right) = 3.6\ \boldsymbol{kip \cdot ft}$$

$$M_{wfront} := M_{1front} + M_{2front} + M_{3front} + M_{4front}$$

$$M_{wback} := M_{1back} + M_{2back} + M_{3back} + M_{4back}$$

$$M_w := M_{wfront} + M_{wback} = 52.4\ \boldsymbol{kip \cdot ft}$$

(Table A13.2-1)	Longitudinal length for distribution of impact force:

$$L_t := 3.5\ \boldsymbol{ft}$$

(Table A13.2-1)	Height of barrier:

$$H := 36\ \boldsymbol{in}$$

(A13.3.1-2)	Critical barrier length for interior region:

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8\ H \cdot (M_w)}{M_c}} = 12.93\ \boldsymbol{ft}$$

(A13.3.1-1)	Transverse resistance of barrier:

$$R_w := \left(\frac{2}{2\ L_c - L_t}\right)\left(8\ M_w + \frac{M_c \cdot {L_c}^2}{H}\right) = 89\ \boldsymbol{kip}$$

(Table A13.2-1)	F_t=54.0 kip, which is the required transverse capacity for a TL-4 barrier from AASHTO Table A13.2-1
(1.3.2.1)	Extreme event phi factor: $\phi := 1.0$

$$\phi R_w := \phi \cdot R_w = 89\ \boldsymbol{kip}$$

$$\phi R_w = 89\ \boldsymbol{kip} > \ F_t := 54\ \boldsymbol{kip}$$

The interior barrier region provides adequate moment capacity

F_t =54 kips is applied at a height of 32" and ϕR_w was calculated at the top of the 36" barrier. A more accurate method involves comparing ϕR_w to an equivalent F_t at the top of the barrier. This is done assuming the moment about the base of the barrier stays constant. For example, 54 kips * 32" = 1,728 k-in. / 36" = 48 kips, which is the equivalent F_t at the top of the barrier.

Barrier end region:

(13.3.1)	Determine the flexural resistance of cantilevered railing about an axis parallel to the longitudinal axis of the bridge for an end barrier region (M_c).

The elevation of the end region retrofit barrier with reinforcement and post-installed rebar is shown in Figure 10.

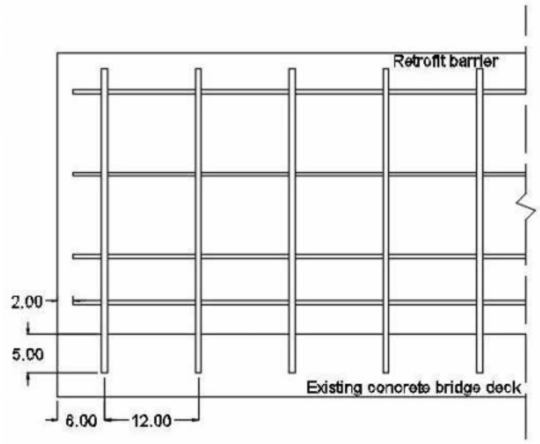


Figure 10. Elevation view of retrofit barrier end region (units of in.)

Design assumptions:

For this example #6 rebar is post-installed

Rebar properties: $d_{a\#6} := 0.75 \cdot in$ $A_{s\#6} := 0.44\ in^2$

$f_y := 60\ ksi$ $f_u := 90\ ksi$

Effective embedment: $h_{ef} := 5 \cdot in$

The effective embedment depth of 5 in. was chosen to avoid interferance with the bottom layer of deck rebar.

(ACI 17.2.7) Concrete compressive strength: $f'_c := 4000\ psi$

Note that ACI 318 equations require f'_c to be given in psi.

Unit weight of concrete: $w_c := 150\ pcf$

(ACI 19.2.4.2) Lightweight concrete modification factor: $\lambda_a := 1.0$ For normal weight concrete

Post-installed rebar spacing: $rebar\_spa := 12\ in$

Area of post-installed rebar per foot: $A_s := \dfrac{A_{s\#6}}{rebar\_spa} = 0.44\ \dfrac{in^2}{ft}$

Calculate M_c tensile capacity of rebar post-installed with a chemical adhesive in accordance with ACI 318-14.

(6.13.2.10.2) Steel strength in tension:

$$N_{saPerFt} := 0.76 \cdot A_s \cdot f_u = 30.1 \ \frac{kip}{ft}$$

(1.3.2.1) Extreme event phi factor: $\phi := 1.0$

$$\phi N_{saPerFt} := \phi \cdot N_{saPerFt} = 30.1 \ \frac{kip}{ft}$$

(ACI 17.4.2) Concrete breakout strength due to tension on anchor:

(ACI 17.4.2.1c) Projected influence area not considering edge distances:

$$A_{Nco} := 9 \ h_{ef}^{\ 2} = 225 \ in^2$$

(ACI Fig R17.4.2.1) Projected influence area considering edge distances and group effects:

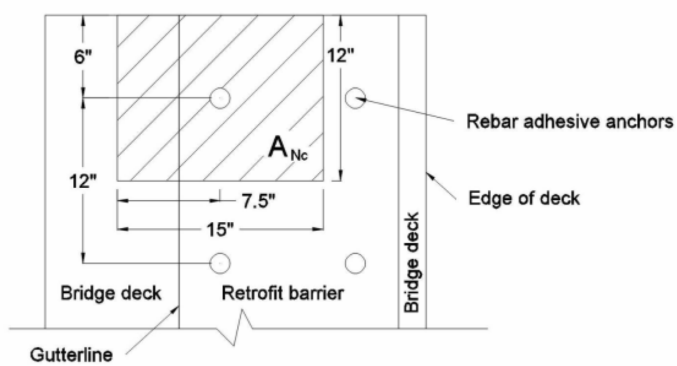


Figure 11. Projected influence area (A_{Nc})

$$rebar\_spa = 12 \ in$$

$$1.5 \cdot h_{ef} = 7.5 \ in \quad > \quad \frac{rebar\_spa}{2} = 6 \ in$$

Post-installed rebar is treated as a group because $1.5\, h_{ef}$ is greater than half the rebar spacing

$$A_{Nc} := \left(2 \cdot 1.5 \cdot h_{ef}\right) \cdot \left(2 \cdot \left(\frac{rebar\_spa}{2}\right)\right) = 180 \ in^2$$

(ACI 17.4.2.2a) Basic concrete breakout strength:

$h_{ef} = 5\ in$ < 11 in., so: $k_c := 17$

$N_b := k_c \cdot \lambda_a \cdot \sqrt{f'_c} \cdot (h_{ef})^{1.5} \cdot \sqrt{lbf \div in} = 12.0\ kip$

(ACI 17.4.2.4) Modification factor for eccentric loading:

$\psi_{ec.N} := 1.0$

(ACI 17.4.2.5) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from center of the rebar to the edge of concrete bridge deck

Post-installed rebar needs 2" clear cover on the traffic face of the barrier and the rebar needs to be straight for 3" before being bent, see Figure 8 for $c_{a.min}$.

$c_{a.min} := 15\ in$

$c_{a.min} = 15\ in$ > $1.5 \cdot h_{ef} = 7.5\ in$ so, $\psi_{ed.N} := 1.0$

(ACI 17.4.2.6) Modification factor for cracking:

Assume that the deck concrete is cracked

$\psi_{c.N} := 1.0$

(ACI 17.4.2.7) Modification factor for splitting:

Modification factor is not considered because it was assumed that the deck concrete is cracked.

(ACI 17.4.2.1b) Concrete breakout strength in tension for a group of post installed-rebar:

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot N_b = 9.6\ kip$$

Concrete breakout strength resistance reduction factor:

(1.3.2.1) Extreme event phi factor: $\phi_{cb} := 1.0$

Unfactored concrete breakout strength in tension per foot:

$$N_{cb.PerFt} := N_{cb} \cdot \frac{1}{rebar\_spa} = 9.6\ \frac{kip}{ft}$$

Factored concrete breakout strength in tension per foot:

$$\phi N_{cb.PerFt} := \phi_{cb} \cdot N_{cb.PerFt} = 9.6 \ \frac{kip}{ft}$$

(ACI 17.4.5) Bond strength:

It is assumed that the concrete bridge deck will be uncracked.
MnDOT specifies a bond strength value for uncracked concrete.

(MnDOT, 2016) $\tau_{uncr} := 1000 \cdot psi$

(ACI 17.4.5.1d) Projected distance to develop bond strength:

$$c_{Na} := 10 \cdot d_{a\#6} \cdot \sqrt{\frac{\tau_{uncr}}{1100 \cdot psi}} = 7.15 \ in$$

(ACI 17.4.5.1c and ACI Fig R17.4.5.1) Projected influence area not considering edge distances:

$$A_{Nao} := (2 \cdot c_{Na})^2 = 204.5 \ in^2$$

(ACI Fig R17.4.5.1) Projected influence area considering edge distances and group effects:

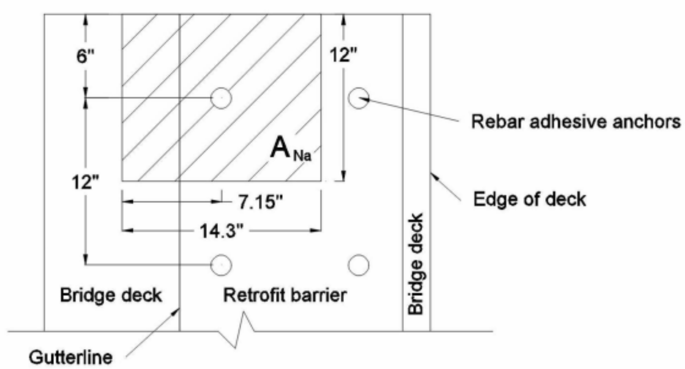


Figure 12. Projected influence area (A_{Na})

Because c_{Na} is greater than half of the post-installed rebar spacing, the rebar is treated as an anchor group for bond strength

$$c_{Na} = 7.15 \ in \quad > \quad \frac{rebar\_spa}{2} = 6 \ in$$

$$A_{Na} := (2 \cdot c_{Na}) \cdot rebar\_spa = 171.6 \ in^2$$

(ACI 17.4.5.2) Basic bond strength:

Concrete is assumed to be uncracked

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_{a\#6} \cdot h_{ef} = 11.8\ \boldsymbol{kip}$

(ACI 17.4.5.4) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from center of the rebar to the edge of concrete

Post-installed rebar needs 2" clear cover on the traffic face of the barrier and the rebar needs to be straight for 3" before being bent, see Figure 8 for $c_{a.min}$.

$c_{a.min} := 15\ \boldsymbol{in}$

$c_{a.min} = 15\ \boldsymbol{in}$ > $c_{Na} = 7.15\ \boldsymbol{in}$ so, $\psi_{ed.Na} := 1.0$

(ACI 17.4.5.5) Modification factor for splitting:

Supplementary reinforcement is provided in the bridge deck

$\psi_{cp.Na} := 1.0$

(ACI 17.4.5.1a) Bond strength in tension per post-installed rebar:

$$N_a := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ed.Na} \cdot \psi_{cp.Na} \cdot N_{ba} = 9.9\ \boldsymbol{kip}$$

Bond strength resistance reduction factor

(1.3.2.1) Extreme event phi factor: $\phi_{ba} := 1.0$

Unfactored bond strength in tension per foot:

$$N_{a.PerFt} := N_a \cdot \frac{1}{rebar\_spa} = 9.9\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

Factored bond strength in tension per foot:

$$\phi N_{a.PerFt} := \phi_{ba} \cdot N_{a.PerFt} = 9.9\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

Barrier resistance:

(A13.3.1) To calculate M_c, the minimum tensile capacity of rebar post-installed with a chemical adhesive was calculated in accordance with ACI 318-14 and AASHTO. The moment capacity is calculated using the internal lever arm and Whitney's equivalent rectangular stress block.

Depth of Whitney's equivalent rectangular stress block per foot calculated using the minimum unfactored tensile capacity from the post-installed rebar design instead of the rebar yield strength:

$$F := min\left(N_{cb.PerFt}, N_{a.PerFt}, N_{saPerFt}\right) = 9.6\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

$$a := \frac{F}{0.85 \cdot f'_c} = 0.236\ \boldsymbol{in/ft}$$

The internal lever arm (d) is shown in Figure 8.

$$d := 13\ \boldsymbol{in}$$

The internal lever arm (d) at the bottom of the barrier height was selected because that is the critical location of the yield-line capacity for post-installed rebar. This value of d matches the assumption made for calculating a single Whitney's equivalent rectangular stress block depth (a) at the bottom of the barrier. For CIP barriers, the internal lever arm (d) is typically averaged over the barrier height.

$$M_c := min\left(\phi N_{cb.PerFt}, \phi N_{a.PerFt}, \phi N_{saPerFt}\right) \cdot \left(d - \frac{a}{2}\right)$$

$$M_c = 10.3\ \frac{\boldsymbol{kip \cdot ft}}{\boldsymbol{ft}}$$

Determine the flexural resistance of the railing about its vertical axis for an end region (M_w).

Steps for calculating M_w for an end region are similar to an interior region, except the horizontal rebar will not be fully developed, which will reduce the M_w. The pocess to calculate M_w is iterative, the L_c needs to be assumed because it effects how much of the horizontal rebar is developed and then the assumed L_c needs to be compared to the calculated L_c.

$$M_w = M_{wfront} + M_{wback}$$

Design assumptions:

Diameter of longitudinal rebar: $d_{b\#4} := 0.5\ \boldsymbol{in}$

Area of longitudinal rebar: $A_{s\#4} := 0.2\ \boldsymbol{in}^2$

Yield strength of rebar: $f_y := 60\ \textbf{ksi}$

Concrete compressive strength: $f'_c := 4\ \textbf{ksi}$

Development length modification factors:

Concrete density modification factor: $\lambda := 1.0$

Coating factor: $\lambda_{cf} := 1.0$

Reinforcement confinement factor: $\lambda_{rc} := 1.0$

Excess reinforcement factor: $\lambda_{cr} := 1.0$

Development length for #4 rebar with greater than or equal to 12" of concrete below:

Reinforcement location factor with 12" of fresh concrete below: $\lambda_{rl} := 1.3$

(5.10.8.2.1)

$$l_{d\#4\_g12} := 2.4 \cdot d_{b\#4} \cdot \frac{\left(\frac{f_y}{\textbf{ksi}}\right)}{\sqrt{\left(\frac{f'_c}{\textbf{ksi}}\right)}} \cdot \left(\frac{\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{cr}}{\lambda}\right) = 46.8\ \textbf{in}$$

Development length for #4 rebar with less than 12" of concrete below:

Reinforcement location factor with less than 12" of fresh concrete below: $\lambda_{rl} := 1.0$

(5.10.8.2.1)

$$l_{d\#4} := 2.4 \cdot d_{b\#4} \cdot \frac{\left(\frac{f_y}{\textbf{ksi}}\right)}{\sqrt{\left(\frac{f'_c}{\textbf{ksi}}\right)}} \cdot \left(\frac{\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{cr}}{\lambda}\right) = 36\ \textbf{in}$$

Moments calculated using traffic side rebar as shown in Figure 8

$d_{1front} := 12.18\ \textbf{in}$ $\quad b_{1front} := 7.00\ \textbf{in}$
$d_{2front} := 11.01\ \textbf{in}$ $\quad b_{2front} := 8.25\ \textbf{in}$ $\quad$ (1=bottom rebar row, 4=top rebar row)
$d_{3front} := 8.95\ \textbf{in}$ $\quad b_{3front} := 10.50\ \textbf{in}$
$d_{4front} := 6.90\ \textbf{in}$ $\quad b_{4front} := 10.25\ \textbf{in}$

$$M_{1front} := A_{s\#4} \cdot f_y \cdot \left(d_{1front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{1front}}\right) = 11.9\ \textbf{kip} \cdot \textbf{ft}$$

$$M_{2front} := A_{s\#4} \cdot f_y \cdot \left(d_{2front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{2front}} \right) = 10.8 \ \boldsymbol{kip \cdot ft}$$

$$M_{3front} := A_{s\#4} \cdot f_y \cdot \left(d_{3front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{3front}} \right) = 8.8 \ \boldsymbol{kip \cdot ft}$$

$$M_{4front} := A_{s\#4} \cdot f_y \cdot \left(d_{4front} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{4front}} \right) = 6.7 \ \boldsymbol{kip \cdot ft}$$

Moments calculated using non-traffic side rebar as shown in Figure 8. Although the top bar has an additional 2" from coping, it is not considered for the moment capacity.

$d_{1back} := 3.75 \ \boldsymbol{in}$ $\quad b_{1back} := 7.00 \ \boldsymbol{in}$

$d_{2back} := 3.75 \ \boldsymbol{in}$ $\quad b_{2back} := 8.25 \ \boldsymbol{in}$ $\quad$ (1=bottom rebar row,

$d_{3back} := 3.75 \ \boldsymbol{in}$ $\quad b_{3back} := 10.50 \ \boldsymbol{in}$ $\quad$ 4=top rebar row)

$d_{4back} := 3.75 \ \boldsymbol{in}$ $\quad b_{4back} := 10.25 \ \boldsymbol{in}$

$$M_{1back} := A_{s\#4} \cdot f_y \cdot \left(d_{1back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{1back}} \right) = 3.5 \ \boldsymbol{kip \cdot ft}$$

$$M_{2back} := A_{s\#4} \cdot f_y \cdot \left(d_{2back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{2back}} \right) = 3.5 \ \boldsymbol{kip \cdot ft}$$

$$M_{3back} := A_{s\#4} \cdot f_y \cdot \left(d_{3back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{3back}} \right) = 3.6 \ \boldsymbol{kip \cdot ft}$$

$$M_{4back} := A_{s\#4} \cdot f_y \cdot \left(d_{4back} - \frac{A_{s\#4} \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b_{4back}} \right) = 3.6 \ \boldsymbol{kip \cdot ft}$$

Moment reduction for development length:

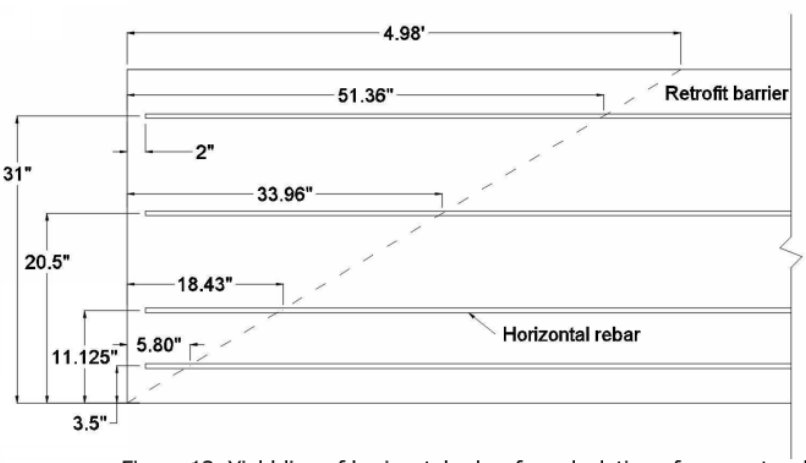


Figure 13. Yield-line of horizontal rebar for calculation of moment reduction

$h_1 := 3.5\ in$
$h_2 := 11.125\ in$
$h_3 := 20.5\ in$
$h_4 := 31\ in$

(1=bottom rebar row,
4=top rebar row)

Need to assume L_c and iterate:

$L_{c\_g} := 4.97\ ft$

(Table A13.2-1) Height of barrier:

$H := 36\ in$

The moment reduction factor for each rebar layer due to development length is calculated using similar triangles with the assumed L_c divided by H and the length of rebar inside the yield-line minus 2" clear cover divided by the height of the horizontal rebar layer.The length of rebar inside the yield-line was calculated and divided by the development length to get the moment reduction factor.

$$r_{h1} := \frac{\left(\frac{h_1 \cdot L_{c\_g}}{H}\right) - 2\ in}{l_{d\#4}} = 0.106$$

$$r_{h2} := \frac{\left(\frac{h_2 \cdot L_{c\_g}}{H}\right) - 2\ in}{l_{d\#4\_g12}} = 0.351$$

(1=bottom rebar row,
4=top rebar row)

$$r_{h3} := \frac{\left(\frac{h_3 \cdot L_{c\_g}}{H}\right) - 2\ in}{l_{d\#4\_g12}} = 0.683$$

$$r_{h4} := \frac{\left(\frac{h_4 \cdot L_{c\_g}}{H}\right) - 2\ in}{l_{d\#4\_g12}} = 1.055$$ > 1.0, so use $r_{h4} := 1.0$

$$M_{wfront} := r_{h1} \cdot M_{1front} + r_{h2} \cdot M_{2front} + r_{h3} \cdot M_{3front} + r_{h4} \cdot M_{4front}$$

$$M_{wback} := r_{h1} \cdot M_{1back} + r_{h2} \cdot M_{2back} + r_{h3} \cdot M_{3back} + r_{h4} \cdot M_{4back}$$

$$M_w := M_{wfront} + M_{wback} = 25.4\ kip \cdot ft$$

Table A13.2-1 Longitudinal length of distribution of impact force:

$L_t := 3.5\ \boldsymbol{ft}$

(A13.3.1-4) Critical barrier length for end region:

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H \cdot \left(\frac{M_w}{M_c}\right)} = 4.98\ \boldsymbol{ft}$$

$L_{c\_g} = 4.97\ \boldsymbol{ft}$ The assumed iterated L_{c_g} is close to actual L_c

(A13.3.1-3) Transverse resistance of barrier:

$$R_w := \left(\frac{2}{2\ L_c - L_t}\right) \left(M_w + \frac{M_c \cdot L_c^{\ 2}}{H}\right) = 34\ \boldsymbol{kip}$$

(Table A13.2-1) F_t =54.0 kip, which is the required transverse capacity for a TL-4 barrier from AASHTO Table A13.2-1

(1.3.2.1) Extreme event phi factor: $\phi := 1.0$

$\phi R_w := \phi \cdot R_w = 34\ \boldsymbol{kip}$

$\phi R_w = 34\ \boldsymbol{kip} < F_t := 54\ \boldsymbol{kip}$

The barrier end region does not provide the required capacity.

F_t =54 kips is applied at a height of 32" and ϕR_w was calculated at the top of the 36" barrier. A more accurate method involves comparing ϕR_w to an equivalent F_t at the top of the barrier. This is done assuming the moment about the base of the barrier stays constant. For example, 54 kips * 32" = 1,728 k-in. / 36" = 48 kips, which is the equivalent F_t at the top of the barrier.

In this example, post-installed rebar can not provide the required moment capacity in the barrier end region. Therefore, other options to suppliment the capacity should be explored, including using deeper h_{ef}, closer rebar spacing, larger bar size, or cast-in-place (CIP) rebar. In some cases CIP is the only viable option, and this method will be used in this example. Part of the end region concrete deck overhang will be torn out to expose existing rebar from the concrete deck overhang, and the reinforcement required to attach the retrofit barrier will be cast into a new concrete deck overhang in the end regions.

CIP barrier end region:

Because a CIP barrier is a standard shape, no further analysis is needed. Standard MnDOT Type S barriers satisfy the requirements for both MASH TL-4 and NCHRP 350 TL-4. The CIP barrier end region should extend over a length of at least L_c.

Barrier shear design:

Only the end barrier region will be designed for shear because it is the worst case (the end region has the shortest effective shear width to resist shear demand). First, shear will be checked using only interface shear between the barrier and the concrete deck. The rebar shear strength will not be used unless more capacity is required.

Demand:

MnDOT policy uses an effective shear width equal to 45° from the minimum height of horizontal loads (H_e) to the gutter plus the distribution of impact force length (L_t).

Minimum height of horizontal loads: $H_e := 32\ in$

Longitudinal length of distribution of impact force: $L_t = 42\ in$

Effective shear width: $L_s := H_e \cdot \cos(45°) + L_t = 5.4\ ft$

(Table A13.2-1) Transverse force acting on barrier: $F_t := 54\ kip$

Interface shear demand per foot: $V_{ui} := \dfrac{F_t}{L_s} = 10\ \dfrac{kip}{ft}$

Capacity:

Only interface shear will be considered on a per foot basis. Assume the original barrier was saw-cut from the deck and that the surface where the new barrier is attached was not intentionally roughened. Interface shear assumptions include normal weight concrete placed on a clean concrete surface, free of laitance, with the surface not intentionally roughened. The rebar shear strength will not be used unless more capacity is required.

(5.7.4) Cohesion factor, friction factor, fraction of concrete strength available to resist interface shear, limiting interface shear resistance:

Cohesion factor: $c := 0.075\ ksi$

Friction factor: $\mu := 0.6$

Fraction of concrete strength available to resist interface shear: $K_1 := 0.2$

Limiting interface shear resistance: $K_2 := 0.8\ ksi$

(5.7.4.3-6) Area of concrete in interface shear per foot:

Width of barrier: $b := 18\ in$

Length of barrier: $l := 12\ in$

$$A_{cv} := \frac{b \cdot l}{ft} = 216\ \frac{in^2}{ft}$$

Permanent net compressive force:

Weight of Type S barrier from Bridge Details Part II FIG. 5-397.138(A)

(MnDOT, 2018) $P_c := 0.496\ \frac{kip}{ft}$

(5.7.4.3) Shear resistance is the minimum of the following shear capacities:

(5.7.4.3-3) $V_{ni.1} := c \cdot A_{cv} + \mu \cdot \left(P_c\right) = 16.5\ \frac{kip}{ft}$

(5.7.4.3-4) $V_{ni.2} := K_1 \cdot f'_c \cdot A_{cv} = 172.8\ \frac{kip}{ft}$

(5.7.4.3-5) $V_{ni.3} := K_2 \cdot A_{cv} = 172.8\ \frac{kip}{ft}$

$$V_{ni} := min\left(V_{ni.1}, V_{ni.2}, V_{ni.3}\right) = 16.5\ \frac{kip}{ft}$$

Factored shear resistance:

(5.5.4.2.1) $\phi := 0.9$

$$\phi V_{ri} := \phi \cdot V_{ni} = 15\ \frac{kip}{ft}$$

Shear demand vs. shear capacity:

Shear capacity = $\phi V_{ri} = 15\ \frac{kip}{ft}$

Shear demand = $V_{ui} = 10\ \frac{kip}{ft}$

$\phi V_{ri} = 15\ \frac{kip}{ft}$ > $V_{ui} = 10\ \frac{kip}{ft}$

The barrier provides the required shear capacity considering only interface shear in an end barrier region. No other regions will be checked for shear capacity because the end barrier region is the worst shear case (the end region has the shortest L_c to resist shear demand).

Summary:

In this example, a retrofit barrier is attached to an existing concrete bridge deck using rebar post-installed with a chemical adhesive. The barrier used was a MnDOT standard 36 in. Type S, TL-4. The geometry of the barrier and the rebar sizes and types were illustrated in Figure 1 of this example. The barrier design followed the method presented in the 8th Edition of AASHTO in Section A13.3.1. The post-installed rebar design followed Ch. 17 of ACI 318-14, except for steel strength, which followed AASHTO Section 6.13.2.11.

The interior barrier region met the flexural capacity requirements for a TL-4 barrier using rebar post-installed with a chemical adhesive spaced at 12 in. on center. The end barrier region did not meet the flexural requirements for a TL-4 barrier using post-installed rebar, so CIP rebar was used instead. Using CIP rebar, the end region Type S barrier meets the flexural capacity requirements.

The worst case barrier end region interface shear resistance was adequate when compared to the shear demand in this example. The rebar shear capacity was neglected.

References

ACI Committee 318. (2014). *Building code requirements for structural concrete (ACI 318-14) and commentary.* Farmington Hills, MI.

American Association of State and Highway Transportation Officials (AASHTO). (2017). *AASHTO LRFD bridge design specifications 8th edition*. Washington, D.C.

MnDOT. (2016). *Approved Concrete Adhesive Anchorage Program*. Oakdale, MN.

MnDOT. (2018). *Bridge Details Manual Part II.* Oakdale, MN.

National Cooperative Highway Research Program (NCHRP). (1993). *Recommended Procedures for the Safety Performance Evaluation of Highway Features (NCHRP 350).* Washington, DC: H. Ross, D. Sicking, and R. Zimmer.

Crash strut retrofit design example

In this example, a retrofit crash strut is post-installed into the pier column footings at a two-column pier using rebar post-installed with a chemical adhesive. The geometry of the crash strut is shown in Figures 1 and 2, which follows guidance from Section 11.2.3.2.4 of the MnDOT Bridge Design Manual (BDM). The load demand is from BDM Section 11.2.3.2.4. There are two design cases to consider when using yeild-line theory to design a crash strut. Case 1 assumes a diagonal yield-line and Case 2 assumes a horizontal yield-line located at the footing-to-crash strut interface. This example will only examine Case 2, which typically provides the minimum moment capacity. The post-installed rebar design follows the requirements from ACI 318-14 Ch. 17, except for the steel strength, which follows AASHTO Section 6.13.2.11. The post-installed rebar layout is shown in Figure 3.

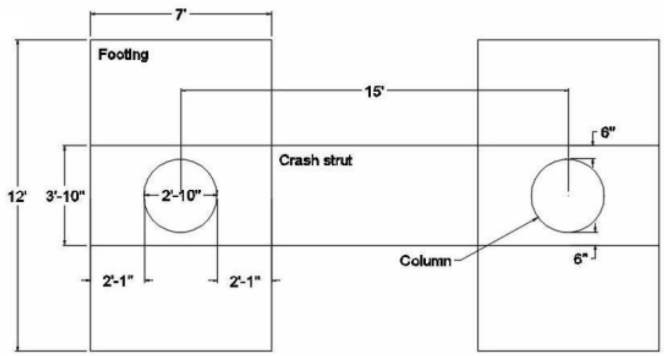


Figure 1. Crash strut plan view

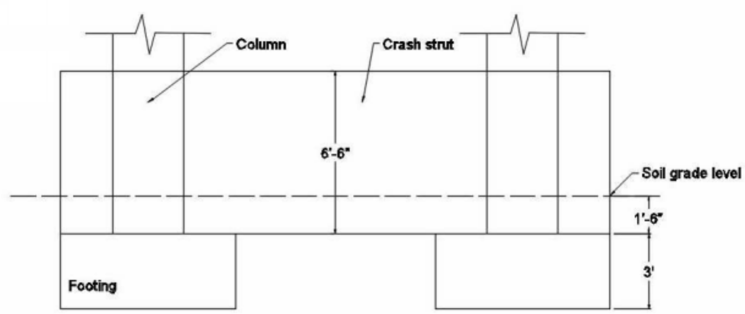


Figure 2. Crash strut elevation view

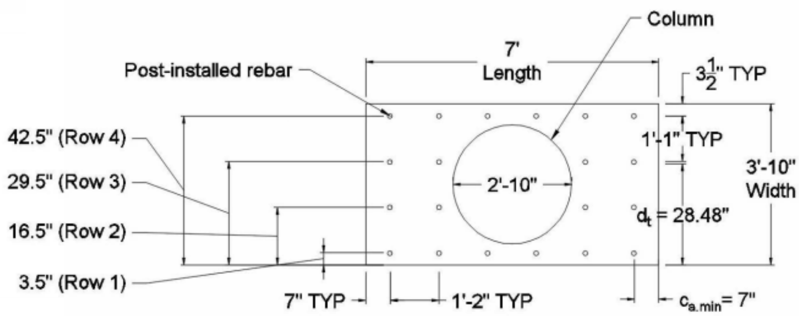


Figure 3. Post-installed rebar layout

Crash strut load demand:

(BDM 11.2.3.2.4) Assuming Case 2 from MnDOT BDM:

(BDM 11.2.3.2.1) Collision load: $P_{crash} := 600\ \boldsymbol{kip}$

Collision load angle: $\theta_{crash} := 15\ \boldsymbol{deg}$

Horizontal collision load: $P_u := P_{crash} \cdot \sin\left(\theta_{crash}\right) = 155\ \boldsymbol{kip}$

Height of crash strut: $h_{strut} := 6.5\ \boldsymbol{ft}$

Moment at base of crash strut: $M_u := P_u \cdot h_{strut} = 1009\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

Shear at base of crash strut: $V_u := P_u = 155\ \boldsymbol{kip}$

Moment capacity per footing:

Design assumptions:

Post-installed rebar spacing is shown in Figure 3.

Rebar properties: $d_{a\#6} := 0.75 \cdot \boldsymbol{in}$ $\quad A_{s\#6} := 0.44\ \boldsymbol{in}^2$

$f_y := 60\ \boldsymbol{ksi}$ $\quad f_u := 90\ \boldsymbol{ksi}$

$E_s := 29000\ \boldsymbol{ksi}$ $\quad \varepsilon_y := \dfrac{f_y}{E_s} = 0.00207$

Effective embedment: $h_{ef} := 14 \cdot \boldsymbol{in}$

Concrete compressive strength: $f'_c := 4000\ \boldsymbol{psi}$

Note that ACI 318 equations require f'_c to be given in psi.

Unit weight of concrete: $w_c := 150\ pcf$

Lightweight concrete modification factor: $\lambda_a := 1.0$ For normal weight concrete

Distance to centerline of rebar row as measured from face of crash strut:

Depth to first row of post-installed rebar: $d_1 := 3.5\ in$

Number of post-installed rebar in first row: $n_1 := 6$

Depth to second row of post-installed rebar: $d_2 := 16.5\ in$

Number of post-installed rebar in second row: $n_2 := 4$

Depth to third row of post-installed rebar: $d_3 := 29.5\ in$

Number of post-installed rebar in third row: $n_3 := 4$

Depth to fourth row of post-installed rebar: $d_4 := 42.5\ in$

Number of post-installed rebar in fourth row: $n_4 := 6$

Area of rebar:

Ignore the outer row of rebar on the compression side (row 1) because row 1 rebar can be in tension or compression and its capacity is minimal.

$$A_s := \left(n_2 + n_3 + n_4\right) \cdot A_{s\#6} = 6.16\ in^2$$

This example will not consider the tensile strength of rebar in row 1 when calculating moment capacity of the crash strut.

Calculation of Whitney's equivalent rectangular stress block is calculated using an iterative process to compare the assumed unfactored tensile capacity (F) to the minimum unfactored tensile capacity from the post-installed rebar design. The process was used instead of using the rebar yield strength.

Lengh of crash strut: $l_{cs} := 7\ ft$

Width of crash strut: $b := 46\ in$

Assumed unfactored tensile capacity: $F := 109\ kip$

$$a := \frac{F}{0.85 \cdot f'_c \cdot l_{cs}} = 0.38\ in \qquad \beta_1 := 0.85$$

$$c := \frac{a}{\beta_1} = 0.45 \ \textit{in}$$

The location of resulting tension force in post-installed rebar is shown in Figure 4:

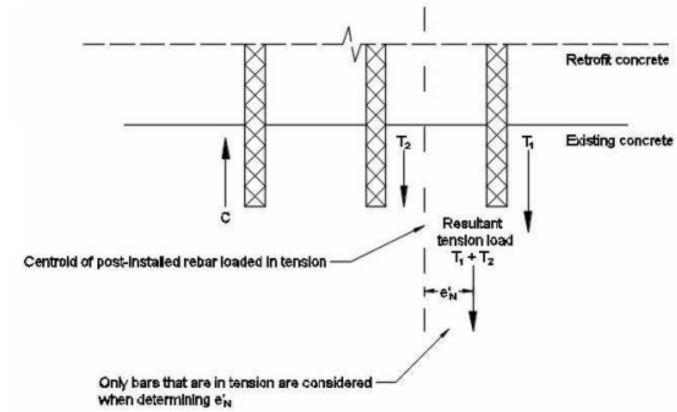


Figure 4. Eccentric loading on group of post-installed rebar

Calculation of d_t which is the location where the resultant tension load is applied with respect to the concrete extreme compression fiber (refer to Figure 3 for dimensions):

$$d_t := \frac{2}{3} \cdot (b - d_1 - c) + c = 28.48 \ \textit{in}$$

Center of gravity location of post-installed rebar in tension:

Ignoring the outer row of rebar on the compression side (row 1) and with respect to the extreme compression fiber (refer to Figure 3 for d_i dimensions).

$$d_{cg} := \frac{n_4 \cdot d_4 + n_3 \cdot d_3 + n_2 \cdot d_2}{n_4 + n_3 + n_2} = 31.4 \ \textit{in}$$

Calculation of e'_N (Figure 4) for use with eccentric loading:

$$e'_N := \left| d_t - d_{cg} \right| = 2.9 \ \textit{in}$$

To calculate the moment capacity, three tensile capacities are calculated, which include steel strength, concrete breakout strength, and bond strength. Illustrations of the three tensile capacities are shown in Figure 5. ACI-318 Table 17.3.1.1 also presents pullout strength in tension (ACI 17.4.3) and concrete side-face blowout strength in tension (ACI 17.4.4) as tensile capacities to be checked, however these tensile capacities are not applicable to post-installed rebar.

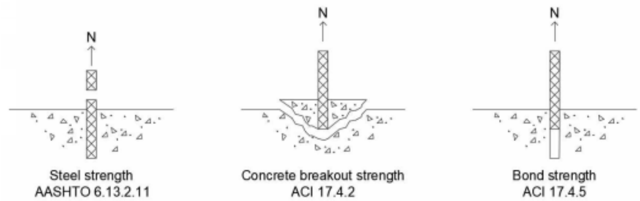


Figure 5. Tensile capacity failure modes

(6.13.2.11) Steel strength:

Assuming only A_s of steel in tension (rebar rows 2-4)

$$N_{sa} := 0.76 \cdot A_s \cdot f_u = 421.3 \ \textbf{kip}$$

(5.5.5) Extreme event phi factor $\phi := 1.0$

$$\phi N_{sa} := \phi \cdot N_{sa} = 421.3 \ \textbf{kip}$$

(ACI 17.4.2) Concrete breakout strength:

(ACI 17.4.2.1c) Projected influence area not considering edge distances:

$$A_{Nco} := 9 \ {h_{ef}}^2 = 1764 \ \textbf{in}^2$$

(ACI 17.4.2.1) Projected influence area considering edge distances and group effects:

Post-installed rebar is treated as a group because $1.5 h_{ef}$ is greater than half the rebar spacing shown in Figure 3.

$$min\_rebar\_spa := 13 \ \textbf{in}$$

$$1.5 \cdot h_{ef} = 21 \ \textbf{in} \quad > \quad \frac{min\_rebar\_spa}{2} = 6.5 \ \textbf{in}$$

To calculate the projected concrete failure area of a group of post-installed rebar, the area of *Area_A*, *Area_B*, and *Area_C* are calculated as shown in Figure 6. *Area_A* extends out of the crash strut into the column foundation. The areas are multiplied by the number of post-installed rebar with the given area to calculate the projected concrete failure area for the group.

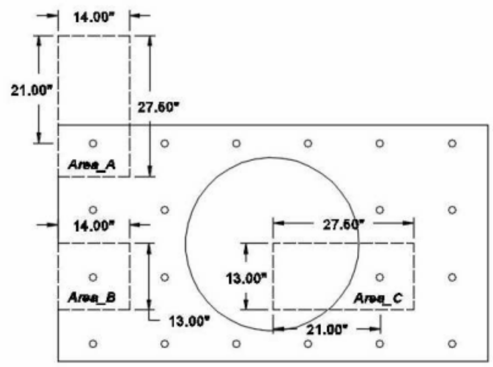


Figure 6. Post-installed rebar area for calculation of A_{Nc}

$$Area\_A := 27.5\ in \cdot 14\ in = 385\ in^2$$

$$Area\_B := 13\ in \cdot 14\ in = 182\ in^2$$

$$Area\_C := 13\ in \cdot 27.5\ in = 357.5\ in^2$$

$$A_{Nc} := 12 \cdot Area\_A + 4 \cdot Area\_B + 4 \cdot Area\_C = 6778\ in^2$$

(ACI 17.4.2.2) Basic concrete breakout strength:

$$h_{ef} = 14\ in$$

(ACI 17.4.2.2) h_{ef} > 11 in., so N_b is the minimumn of $N_{b.1}$ and $N_{b.2}$

(ACI 17.4.2.2a) $$N_{b.1} := \sqrt{\frac{lbf}{in}} \cdot 17 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot (h_{ef})^{1.5} = 56.3\ kip$$

(ACI 17.4.2.2b) $$N_{b.2} := \sqrt{lbf \cdot in} \cdot 16 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot \left(\frac{h_{ef}}{in}\right)^{\frac{5}{3}} = 82.3\ kip$$

$$N_b := min\left(N_{b.1}, N_{b.2}\right) = 56.3\ kip$$

(ACI 17.4.5.3) Modification factor for eccentric loading:

$$\psi_{ec.N} := \frac{1}{\left(1 + \frac{2 \cdot e'_N}{3 \cdot h_{ef}}\right)} = 0.88$$

(ACI 17.4.2.5) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from the center of the rebar to the edge of the concrete as shown in Figure 3

$c_{a.min} := 7\ \boldsymbol{in}$

$c_{a.min} = 7\ \boldsymbol{in}$ < 1.5 $h_{ef} = 14\ \boldsymbol{in}$ so,

$$\psi_{ed.N} := 0.7 + 0.3 \cdot \frac{c_{a.min}}{1.5 \cdot h_{ef}} = 0.8$$

(ACI 17.4.2.6) Modification factor for cracking:

Assume that the foundation concrete is cracked

$\psi_{c.N} := 1.0$

(ACI 17.4.2.7) Modification factor for splitting:

Assume that the foundation concrete is cracked

$\psi_{cp.N} := 1.0$

(ACI 17.4.2.1b) Concrete breakout strength in tension for a group of post-installed rebar:

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 152.3\ \boldsymbol{kip}$$

Concrete breakout strength resistance reduction factor:

(5.5.5) Extreme event phi factor: $\phi_{cb} := 1.0$

Factored concrete breakout strength in tension:

$\phi N_{cb} := \phi_{cb} \cdot N_{cb} = 152.3\ \boldsymbol{kip}$

Bond strength:

Bond strength will be checked for both uncracked and cracked concrete conditions. MnDOT specifies a bond strength value for both uncracked and cracked concrete

(MnDOT, 2016) $\tau_{uncr} := 1000 \cdot \boldsymbol{psi}$

$\tau_{cr} := 500 \cdot \boldsymbol{psi}$

Uncracked bond strength:

(ACI 17.4.5.1d) Projected distance to develop bond strength:

$$c_{Na} := 10 \cdot d_{a\#6} \cdot \sqrt{\frac{\tau_{uncr}}{1100 \cdot psi}} = 7.15\ in$$

(ACI 17.4.5.1c) Projected influence area not considering edge distances:

$$A_{Nao} := \left(2 \cdot c_{Na}\right)^2 = 204.5\ in^2$$

(ACI 17.4.5.1) Projected influence area considering edge distances and group effects:

Projected distance to develop bond strength: $c_{Na} = 7.15\ in$

To calculate the projected bond failure area of a group of post-installed rebar, the area of *Area_A*, *Area_B*, and *Area_C* are calculated as shown in Figure 7. The areas are multiplied by the number of post-installed rebar with the given area to calculate the projected bond failure area for the group.

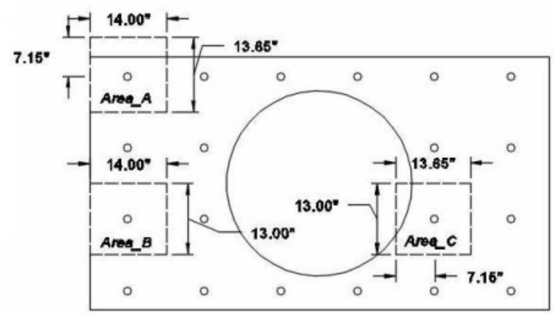


Figure 7. Post-installed rebar area for calculation of A_{Na}

$$Area\_A := 13.651\ in \cdot 14\ in = 191\ in^2$$

$$Area\_B := 13\ in \cdot 14\ in = 182\ in^2$$

$$Area\_C := 13\ in \cdot 13.651\ in = 177\ in^2$$

$$A_{Na} := 12 \cdot Area\_A + 4 \cdot Area\_B + 4 \cdot Area\_C = 3731\ in^2$$

(ACI 17.4.5.2) Basic bond strength:

Concrete is assumed to be uncracked

$$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_{a\#6} \cdot h_{ef} = 33.0\ kip$$

(ACI 17.4.5.3) Modification factor for eccentric loading:

$$\psi_{ec.Na} := \frac{1}{\left(1 + \frac{e'_N}{c_{Na}}\right)} = 0.713$$

(ACI 17.4.5.4) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from the center of the rebar to the edge of the concrete as shown in Figure 3.

$$c_{a.min} := 7\ in$$

$c_{a.min} = 7\ in$ < $c_{Na} = 7.15\ in$ so,

$$\psi_{ed.Na} := 0.7 + 0.3 \cdot \frac{c_{a.min}}{c_{Na}} = 0.994$$

(ACI 17.4.5.5) Modification factor for splitting:

Assume that supplementary reinforcement is not provided in the footing and the concrete is assumed to be uncracked

$c_{a.min}$ is the minimum distance from the center of the rebar to the edge of the concrete as shown in Figure 3

$$c_{a.min} := 7\ in$$

c_{ac} is the critical edge distance required to develop the basic strength controlled by concrete breakout or bond strength

(ACI 17.7.6) $c_{ac} := 2 \cdot h_{ef} = 28\ in$

$c_{a.min} = 7\ in$ < $c_{Na} = 7.15\ in$ so,

$$\frac{c_{a.min}}{c_{ac}} = 0.25$$

$$\frac{c_{Na}}{c_{ac}} = 0.26$$

$$\psi_{cp.Na} := \max\left(\frac{c_{a.min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) = 0.255$$

(ACI 17.4.5.1a): Bond strength in tension per post-installed rebar:

$$N_{una} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec.Na} \cdot \psi_{ed.Na} \cdot \psi_{cp.Na} \cdot N_{ba} = 108.9\ \boldsymbol{kip}$$

Bond strength resistance reduction factor

(5.5.5) Extreme event phi factor $\phi_{ba} := 1.0$

Factored uncracked concrete bond strength in tension:

$$\phi N_{una} := \phi_{ba} \cdot N_{una} = 108.9\ \boldsymbol{kip}$$

Cracked Bond strength:

(ACI 17.4.5.1d) Projected distance to develop bond strength:

$$c_{Na} := 10 \cdot d_{a\#6} \cdot \sqrt{\frac{\tau_{uncr}}{1100 \cdot \boldsymbol{psi}}} = 7.15\ \boldsymbol{in}$$

(ACI 17.4.5.1c) Projected influence area not considering edge distances:

$$A_{Nao} := \left(2 \cdot c_{Na}\right)^2 = 204.5\ \boldsymbol{in}^2$$

(ACI 17.4.5.1) Projected influence area considering edge distances and group effects:

A_{Na} for cracked concrete is the same as A_{Na} is for uncracked concrete

$$A_{Na} = 3731\ \boldsymbol{in}^2$$

(ACI 17.4.5.2) Basic bond strength:

Concrete is assumed to be cracked

$$N_{ba} := \lambda_a \cdot \tau_{cr} \cdot \pi \cdot d_{a\#6} \cdot h_{ef} = 16.5\ \boldsymbol{kip}$$

(ACI 17.4.5.3) Modification factor for eccentric loading:

$$\psi_{ec.Na} := \frac{1}{\left(1 + \frac{e'_N}{c_{Na}}\right)} = 0.713$$

(ACI 17.4.5.4) Modification factor for edge effects:

$c_{a.min}$ is the minimum distance from center of the rebar to the edge of concrete as shown in Figure 3

$c_{a.min} := 7\ in$

$c_{a.min} = 7\ in$ < $c_{Na} = 7.15\ in$ so,

$$\psi_{ed.Na} := 0.7 + 0.3 \cdot \frac{c_{a.min}}{c_{Na}} = 0.99$$

(ACI 17.4.5.5) Modification factor for splitting:

Assume that supplementary reinforcement is not provided to control splitting in the footing and the concrete is assumed to be cracked.

$\psi_{cp.Na} := 1.0$

(ACI 17.4.5.1a) Bond strength in tension per post-installed rebar:

$$N_{cra} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec.Na} \cdot \psi_{ed.Na} \cdot \psi_{cp.Na} \cdot N_{ba} = 213.2\ kip$$

Bond strength resistance reduction factor

(5.5.5) Extreme event phi factor $\phi_{ba} := 1.0$

Factored cracked concrete bond strength in tension:

$\phi N_{cra} := \phi_{ba} \cdot N_{cra} = 213\ kip$

Moment resistance of post-installed rebar:

Assumed unfactored tensile capacity: $F = 109\ kip$

Unfactored tensile capacities:

Steel strength: $N_{sa} = 421\ kip$

Concrete breakout strength: $N_{cb} = 152\ kip$

Uncracked bond strength: $N_{una} = 109\ kip$

Cracked bond strength: $N_{cra} = 213\ kip$

The assumed unfactored tensile capacity (F) was equal to the minimum unfactored tensile capacity from the post-installed rebar design.

Factored tensile capacities:

Steel strength: $\phi N_{sa} = 421\ \mathbf{kip}$

Concrete breakout strength: $\phi N_{cb} = 152\ \mathbf{kip}$

Uncracked bond strength: $\phi N_{una} = 109\ \mathbf{kip}$

Cracked bond strength: $\phi N_{cra} = 213\ \mathbf{kip}$

$$\phi M_{n.footing} := min\left(\phi N_{sa}, \phi N_{cb}, \phi N_{cra}, \phi N_{una}\right) \cdot \left(d_t - \frac{a}{2}\right) = 256.8\ \mathbf{kip \cdot ft}$$

$$\phi M_{n.footing} = 257\ \mathbf{kip \cdot ft} \qquad < \qquad M_u = 1009\ \mathbf{kip \cdot ft}$$

Adequate moment capacity is not provided by the post-installed rebar so the column strength will need to be considered.

Moment demand on column:

The moment demand on the column is the total moment demand minus the moment capacity of the post-installed rebar.

$$M_{u.column} := M_u - \phi M_{n.footing} = 752.6\ \mathbf{kip \cdot ft}$$

Moment capacity of column:

The moment capacity of the column is calculated to determine if the column can take the rest of the load from the crash. The super structure above the pier is unknown, so the axial compressive force in the column is ignored. When information about the super structure is known then use the axial compressive force. Using the axial compressive force with a column interaction diagram will increase the moment capacity that the column can handle in the event of a vehicle collision. The geometry of the column is shown in Figure 8.

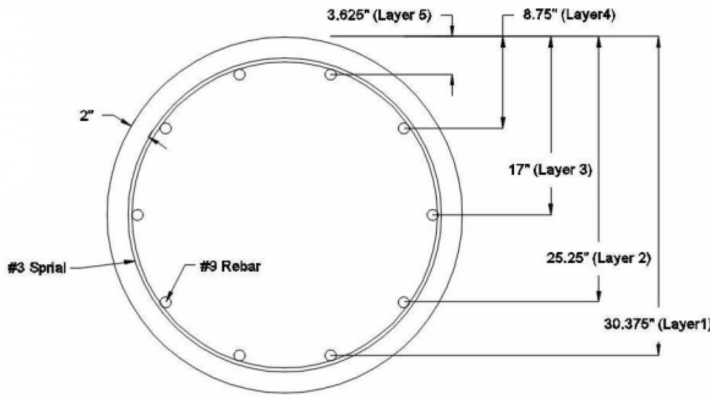


Figure 8. Geometry of column reinforcement

Design assumptions:

Diameter of column: $h_{col} := 34\ \boldsymbol{in}$

Rebar properties: $A_{s\#9} := 1.128\ \boldsymbol{in}^2$

$f_y := 60\ \boldsymbol{ksi}$

$E_s := 29000\ \boldsymbol{ksi}$

$\varepsilon_y := \dfrac{f_y}{E_s} = 0.00207$

Concrete properties: $f'_c = 4\ \boldsymbol{ksi}$ $\beta_1 := .85$

Column capacity:

Distance to layer of rebar closest to tension face:

$d_t := 30.375\ \boldsymbol{in}$

Depth to neutral axis:

$$c := \frac{0.003}{0.003 + \varepsilon_y} \cdot d_t = 17.98\ \boldsymbol{in}$$

Depth of compression block:

$a := \beta_1 \cdot c = 15.28\ \boldsymbol{in}$

Angle of compression block in radians:

$$\alpha := \operatorname{acos}\left(\left(\frac{h_{col}}{2} - a\right) \div \left(\frac{h_{col}}{2}\right)\right) = 1.47$$

Area of compression block:

$$A_{comp} := \frac{h_{col}^{\ 2}}{2} \left(\frac{\alpha}{2} - \frac{\sin(2\ \alpha)}{4}\right) = 395.6\ \boldsymbol{in}^2$$

Center of gravity of compression block:

$$Y := \frac{\frac{h_{col}^{\ 3}}{4} \left(\frac{(\sin(\alpha))^3}{3}\right)}{A_{comp}} = 8.15\ \boldsymbol{in}$$

Resultant compressive force in the concrete:

$$C_c := 0.85 \cdot f'_c \cdot A_{comp} = 1345\ \boldsymbol{kip}$$

Strain in rebar levels (1= near tension face, 5= near compression face):
Positive is tension, negative is compression

$$\varepsilon_{s1} := \frac{0.003}{c} (30.375\ \boldsymbol{in} - c) = 0.00207 \quad = \quad \varepsilon_y \quad \text{Yields}$$

$$\varepsilon_{s2} := \frac{0.003}{c} (25.25\ \boldsymbol{in} - c) = 0.00121 \quad < \quad \varepsilon_y \quad \text{Does not yield}$$

$$\varepsilon_{s3} := \frac{0.003}{c} (17\ \boldsymbol{in} - c) = -0.00016 \quad < \quad \varepsilon_y \quad \text{Does not yield}$$

$$\varepsilon_{s4} := \frac{0.003}{c} (8.75\ \boldsymbol{in} - c) = -0.00154 \quad < \quad \varepsilon_y \quad \text{Does not yield}$$

$$\varepsilon_{s5} := \frac{0.003}{c} (3.625\ \boldsymbol{in} - c) = -0.0024 \quad > \quad \varepsilon_y \quad \text{Yields}$$

Stress in rebar levels (1= near tension face, 5= near compression face):
Positive is tension, negative is compression

$$f_{s1} := f_y = 60\ \boldsymbol{ksi}$$

$$f_{s2} := \frac{\varepsilon_{s2}}{\varepsilon_y} \cdot f_y = 35.2\ \boldsymbol{ksi}$$

$$f_{s3} := \frac{\varepsilon_{s3}}{\varepsilon_y} \cdot f_y = -4.7\ \boldsymbol{ksi}$$

$$f_{s4} := \frac{\varepsilon_{s4}}{\varepsilon_y} \cdot f_y = -44.7\ \textbf{ksi}$$

$$f_{s5} := -f_y = -60\ \textbf{ksi}$$

Force in rebar levels (1= near tension face, 5= near compression face):

Positive is tension, negative is compression

$$F_{s1} := f_{s1} \cdot 2\ A_{s\#9} = 135.4\ \textbf{kip}$$

$$F_{s2} := f_{s2} \cdot 2\ A_{s\#9} = 79.4\ \textbf{kip}$$

$$F_{s3} := f_{s3} \cdot 2\ A_{s\#9} = -10.7\ \textbf{kip}$$

$$F_{s4} := f_{s4} \cdot 2\ A_{s\#9} = -100.7\ \textbf{kip}$$

$$F_{s5} := f_{s5} \cdot 2\ A_{s\#9} = -135.4\ \textbf{kip}$$

Axial capacity:

$$P_n := C_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} = 1313\ \textbf{kip}$$

Moment capacity:

Calculated by summing the moment about the column center line

Moments for the rebar layers (1= near tension face, 5= near compression face):

$$M_1 := F_{s1} \cdot \left(30.375\ \textbf{in} - \frac{h_{col}}{2}\right) = 150.9\ \textbf{kip} \cdot \textbf{ft}$$
$$M_2 := F_{s2} \cdot \left(25.25\ \textbf{in} - \frac{h_{col}}{2}\right) = 54.6\ \textbf{kip} \cdot \textbf{ft}$$
$$M_3 := F_{s3} \cdot \left(17\ \textbf{in} - \frac{h_{col}}{2}\right) = 0\ \textbf{kip} \cdot \textbf{ft}$$
$$M_4 := F_{s4} \cdot \left(8.75\ \textbf{in} - \frac{h_{col}}{2}\right) = 69.3\ \textbf{kip} \cdot \textbf{ft}$$
$$M_5 := F_{s5} \cdot \left(3.625\ \textbf{in} - \frac{h_{col}}{2}\right) = 150.9\ \textbf{kip} \cdot \textbf{ft}$$

$$M_n := C_c \cdot \left(\frac{h_{col}}{2} - Y\right) + M_1 + M_2 + M_3 + M_4 + M_5 = 1417\ \textbf{kip} \cdot \textbf{ft}$$

(5.5.5) Extreme event phi factor $\phi := 1.0$

$$\phi M_{n.column} := \phi \cdot M_n = 1417\ \textbf{kip} \cdot \textbf{ft}$$

$\phi M_{n.column} = 1417\ \boldsymbol{kip \cdot ft}$ > $M_{u.column} = 753\ \boldsymbol{kip \cdot ft}$

The column provides the additional required capacity to resist the crash load.

Crash strut horizontal rebar design:

The horizontal rebar is designed to resist a horizontal collision load (P_u). P_u is applied as a point-load on the crash strut at mid-span between the two-column piers. The horizontal rebar design will be treated as a simply supported beam.

Design assumptions:

Length between piers: $l_{btw\_piers} := 15\ \boldsymbol{ft}$

Horizontal collision load: $P_u = 155.3\ \boldsymbol{kip}$

Rebar properties: $A_{s\#6} = 0.44\ \boldsymbol{in}^2$

$f_y := 60\ \boldsymbol{ksi}$

$E_s := 29000\ \boldsymbol{ksi}$

$$\varepsilon_y := \frac{f_y}{E_s} = 0.00207$$

Concrete properties: $f'_c = 4\ \boldsymbol{ksi}$ $\beta_1 := .85$

Crash strut horizontal moment demand:

Moment demand:

$$M_u := \frac{P_u \cdot l_{btw\_piers}}{4} = 582.3\ \boldsymbol{kip \cdot ft}$$

Crash strut horizontal moment capacity:

Area of hoizontal rebar:

(8) #6 rebar will be used for horizontal rebar

$A_{s\_horz} := 8 \cdot A_{s\#6} = 3.52\ \boldsymbol{in}^2$

Depth of compression block:

$$a := \frac{A_{s\_horz} \cdot f_y}{0.85 \cdot f'_c \cdot h_{strut}} = 0.80\ \boldsymbol{in}$$

Depth to neutral axis:

$$c := \frac{a}{\beta_1} = 0.937\ \textbf{in}$$

Depth to horizontal tension rebar:

Horizontal tension rebar will be placed inside the outer rows of post-installed rebar

$$d_{t\_horz} := 41.75\ \textbf{in}$$

Check horizontal rebar for yeilding for determining resistance reduction factor:

$$\varepsilon_{s\_horz} := \frac{0.003}{c}\left(d_{t\_horz} - c\right) = 0.131 \quad > \quad \varepsilon_y = 0.0021 \qquad \text{Yeilds}$$

Unfactored horizontal moment capacity:

$$M_n := A_{s\_horz} \cdot f_y \cdot \left(d_{t\_horz} - \frac{a}{2}\right) = 727.8\ \textbf{kip} \cdot \textbf{ft}$$

Resistance reduction factor:

$$\phi := 0.9$$

Factored horizontal moment capacity:

$$\phi M_n := \phi \cdot M_n = 655\ \textbf{kip} \cdot \textbf{ft}$$

$$\phi M_n = 655\ \textbf{kip} \cdot \textbf{ft} \quad > \quad M_u = 582.3\ \textbf{kip} \cdot \textbf{ft}$$

Adequate moment capacity is provided by the horizontal rebar

Crash strut shear design:

First, shear will be checked using only interface shear between the crash strut and the concrete footing. The rebar shear strength will not be used unless more capacity is required.

Shear demand per foot:

(BDM 11.2.3.2.4) MnDOT policy uses an effective shear width equal to 5 ft

Effective shear width $L_s := 5\ \textbf{ft}$

Interface shear demand per foot: $V_{ui} := \frac{P_u}{L_s} = 31\ \frac{kip}{ft}$

Shear capacity per foot:

Only interface shear will be considered on a per foot basis. Interface shear assumptions include normal weight concrete placed on a clean concrete surface, free of laitance, with the surface not intentionally roughened.

(5.7.4) Cohesion factor, friction factor, fraction of concrete strength available to resist interface shear, limiting interface shear resistance:

Cohesion factor: $c := 0.075\ ksi$

Friction factor: $\mu := 0.6$

Fraction of concrete strength available to resist interface shear: $K_1 := 0.2$

Limiting interface shear resistance: $K_2 := 0.8\ ksi$

(5.7.4.3-6) Area of concrete in interface shear per foot:

Width of crash strut: $b := 46\ in$

Length of crash strut: $l := 12\ in$

$$A_{cv} := \frac{b \cdot l}{ft} = 552\ \frac{in^2}{ft}$$

Area of interface shear reinforcement:

The rebar shear strength will not be used unless more capacity is required

$$A_{vf} := 0\ \frac{in^2}{ft}$$

Permanent net compressive force per foot:

Crash strut weight per foot:

Height of crash strut: $h_{strut} = 6.5\ ft$

$$P_c := w_c \cdot \frac{(b \cdot l \cdot h_{strut})}{ft} = 3.7 \ \frac{kip}{ft}$$

(5.7.4.3) Shear resistance is the minimum of the following shear capacities:

(5.7.4.3-3) $$V_{ni.1} := c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y + P_c) = 43.6 \ \frac{kip}{ft}$$

(5.7.4.3-4) $$V_{ni.2} := K_1 \cdot f'_c \cdot A_{cv} = 441.6 \ \frac{kip}{ft}$$

(5.7.4.3-5) $$V_{ni.3} := K_2 \cdot A_{cv} = 441.6 \ \frac{kip}{ft}$$

$$V_{ni} := min(V_{ni.1}, V_{ni.2}, V_{ni.3}) = 43.6 \ \frac{kip}{ft}$$

Factored shear resistance per foot:

(5.5.4.2.1) $$\phi := 0.9$$

$$\phi V_{ri} := \phi \cdot V_{ni} = 39 \ \frac{kip}{ft}$$

Shear demand vs. shear capacity:

Shear demand: $V_{ui} = 31 \ \frac{kip}{ft}$

Shear capacity: $\phi V_{ri} = 39 \ \frac{kip}{ft}$

$$\phi V_{ri} = 39 \ \frac{kip}{ft} \quad > \quad V_{ui} = 31 \ \frac{kip}{ft}$$

The crash strut provides the required shear capacity considering only interface shear.

Summary:

In this example, a retrofit crash strut was post-installed into the column footings at a two-column pier using rebar post-installed with a chemical adhesive. The geometry of the crash strut was shown in Figures 1 and 2, which followed guidance from Section 11.2.3.2.4 of the MnDOT Bridge Design Manual (BDM). The load demand was from BDM Section 11.2.3.2.4. There were two design cases to consider when using yeild-line theory to design a crash strut. Case 1 assumes a diagonal yield-line and Case 2 assumes a horizontal yield-line located at the footing-to-crash strut interface. This example only examined Case 2, which typically provides the minimum moment capacity. The post-installed rebar design follows the requirements from ACI 318-14 Ch. 17, except for the steel strength, which followed AASHTO Section 6.13.2.11.

It was found that the post-installed rebar in the crash strut would not provide enough moment capacity, so the column moment capacity had to be considered. The combination of moment capacity from the post-installed rebar and the column provided adequate moment capacity compared to moment demand in this example. Counting on the column capacity should be avoided, but in retrofit designs, it may be the only option to provide the required flexural capacity.

The interface shear resistance was adequate when compared to the shear demand in this example. The rebar shear capacity was neglected.

References

ACI Committee 318. (2014). *Building code requirements for structural concrete (ACI 318-14) and commentary.* Farmington Hills, MI.

American Association of State and Highway Transportation Officials (AASHTO). (2017). *AASHTO LRFD bridge design specifications 8th edition*. Washington, D.C.

MnDOT. (2016). *Approved Concrete Adhesive Anchorage Program*. Oakdale, MN.

MnDOT. (2017). *Bridge Design Manual*. Oakdale, MN.

APPENDIX B: APPLIED TENSILE LOAD VS. DISPLACEMENT PLOTS

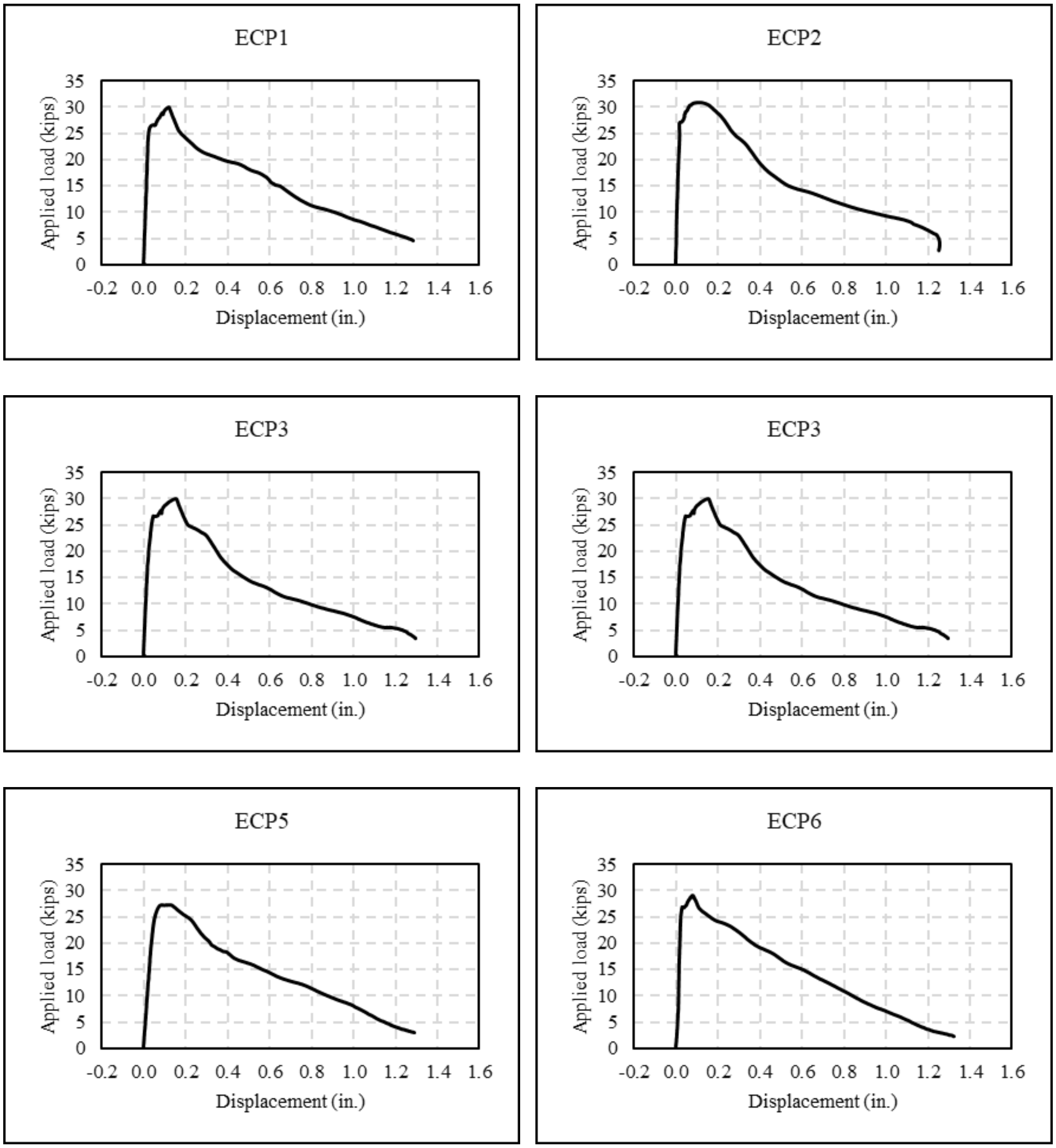


Figure B.1 Applied tensile load vs. displacement plots from test specimens containing epoxy-coated reinforcing bars post-installed with the Powers AC100+ Gold chemical adhesive

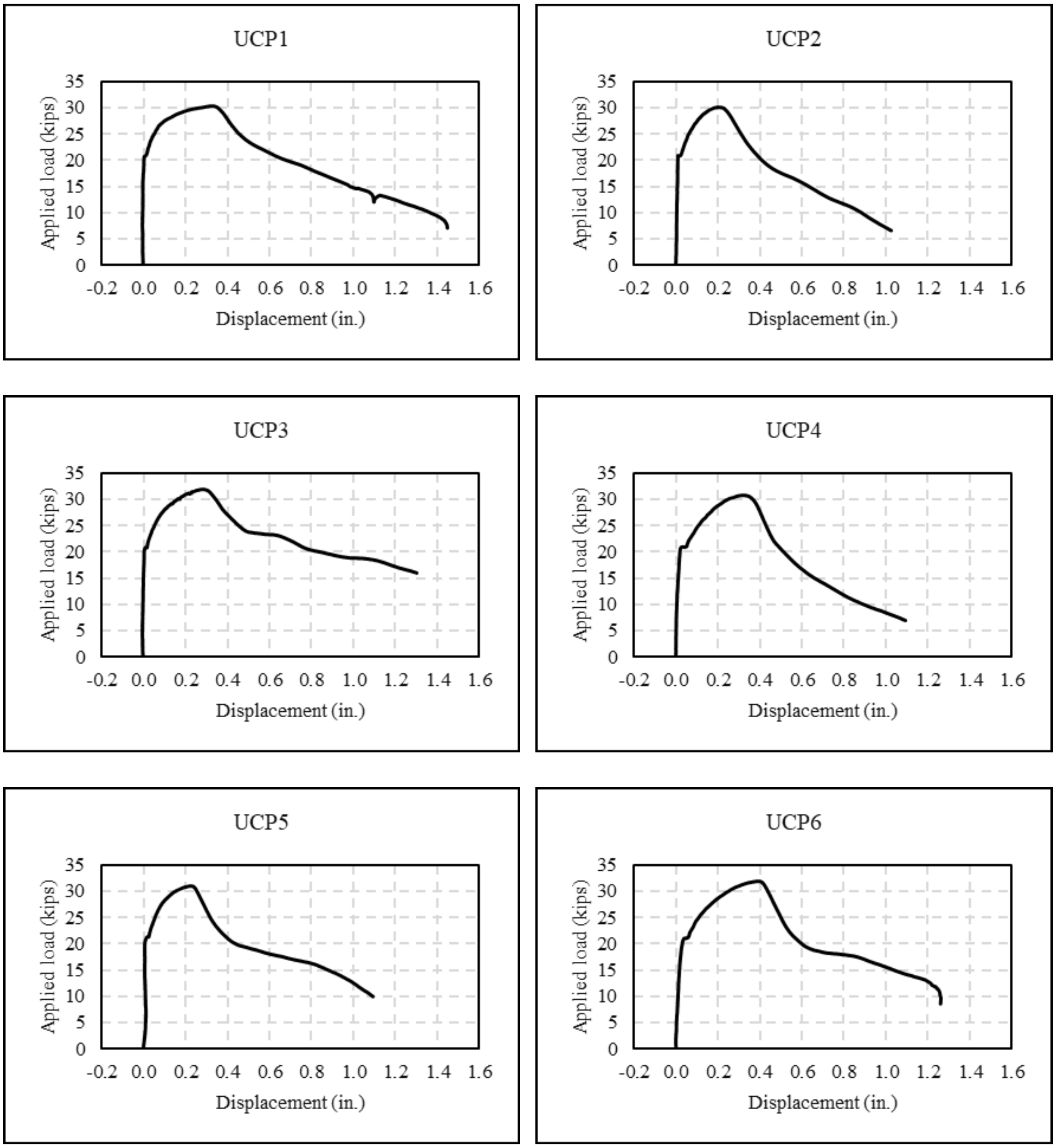


Figure B.2 Applied tensile load vs. displacement plots from test specimens containing uncoated reinforcing bars post-installed with the Powers AC100+ Gold chemical adhesive

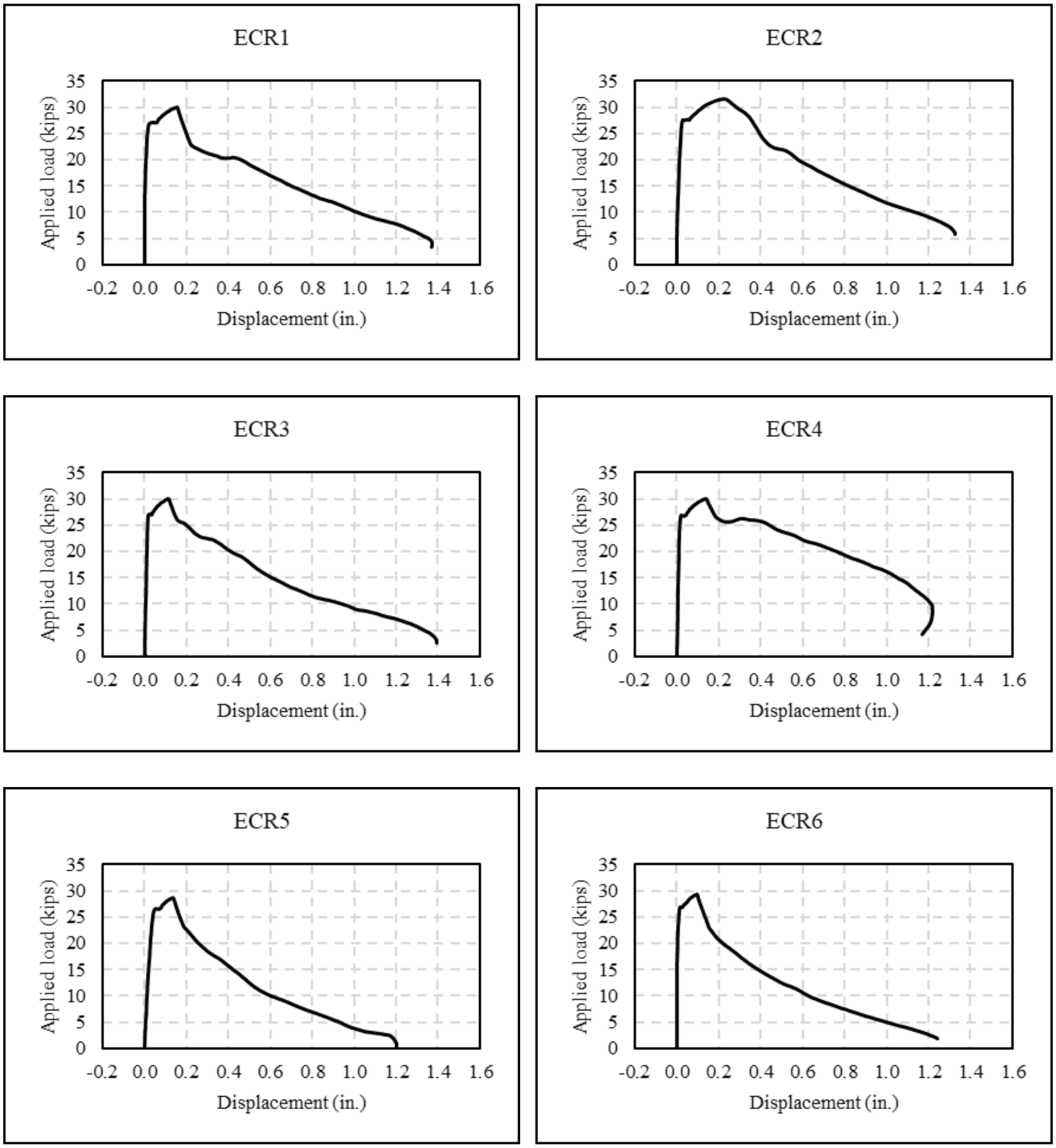


Figure B.3 Applied tensile load vs. displacement plots from test specimens containing epoxy-coated reinforcing bars post-installed with the Red Head A7+ chemical adhesive

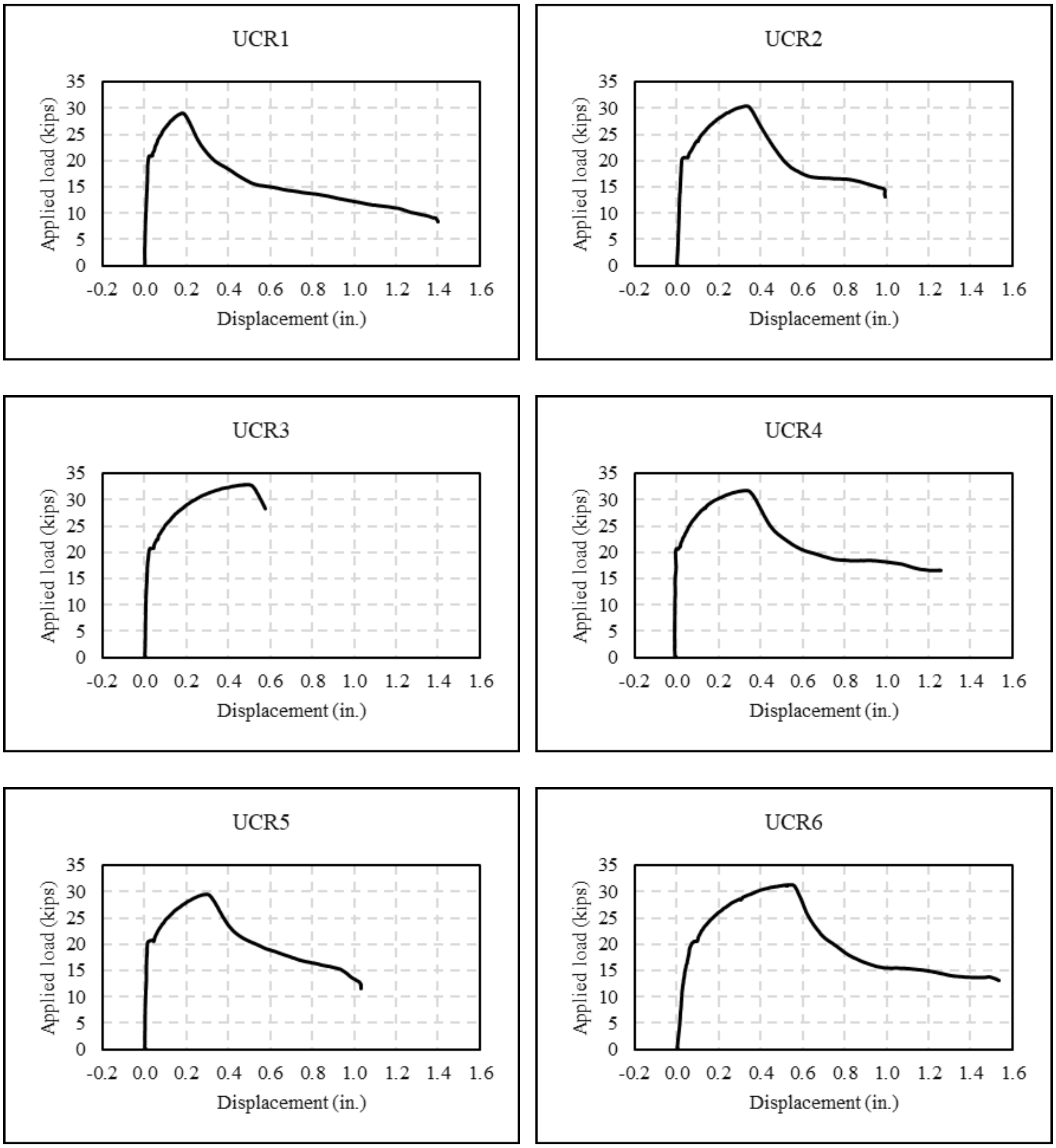


Figure B.4 Applied tensile load vs. displacement plots from test specimens containing uncoated reinforcing bars post-installed with the Red Head A7+ chemical adhesive

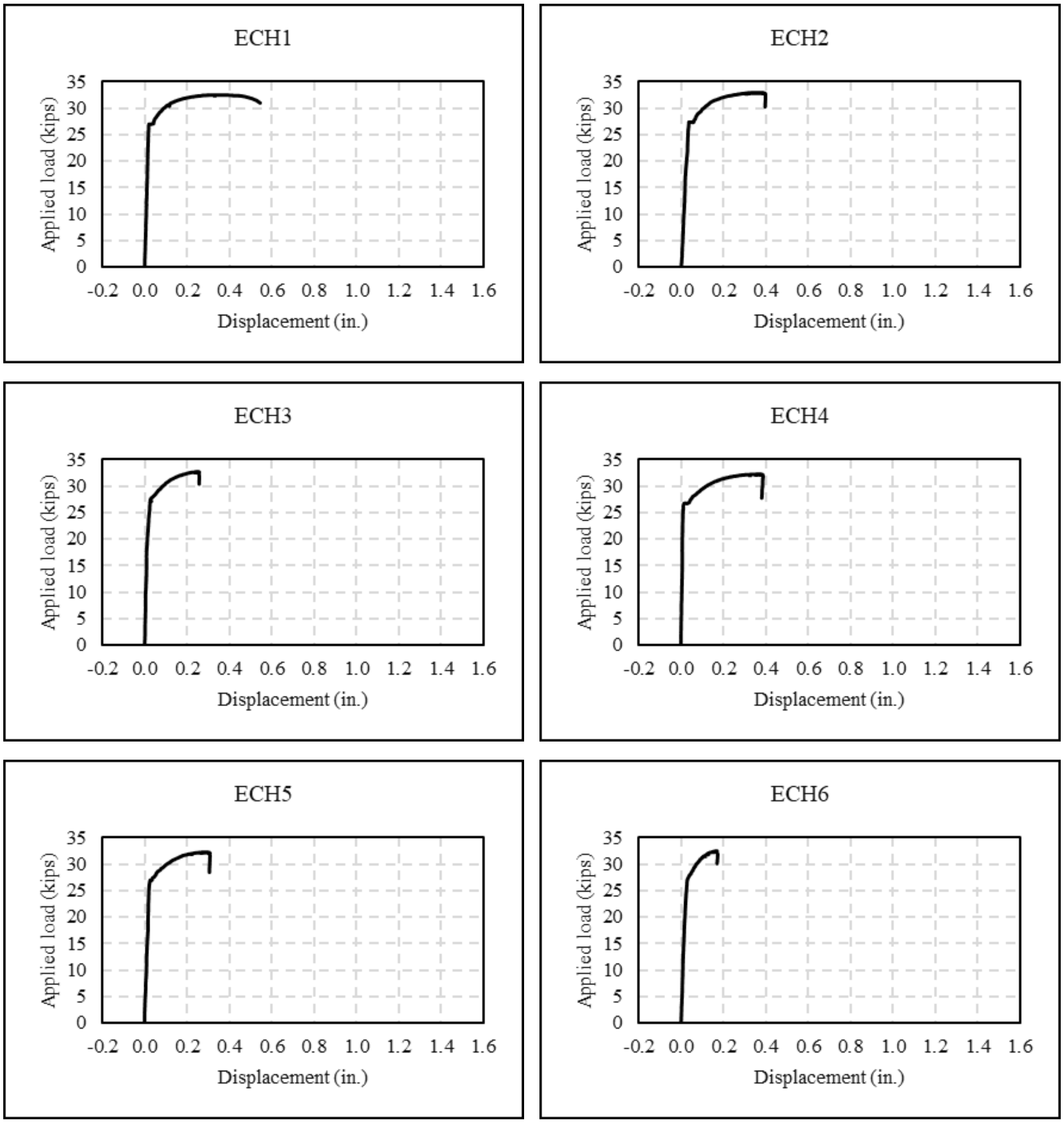


Figure B.5 Applied tensile load vs. displacement plots from test specimens containing epoxy-coated reinforcing bars post-installed with the Hilti HIT-RE 500 V3 chemical adhesive

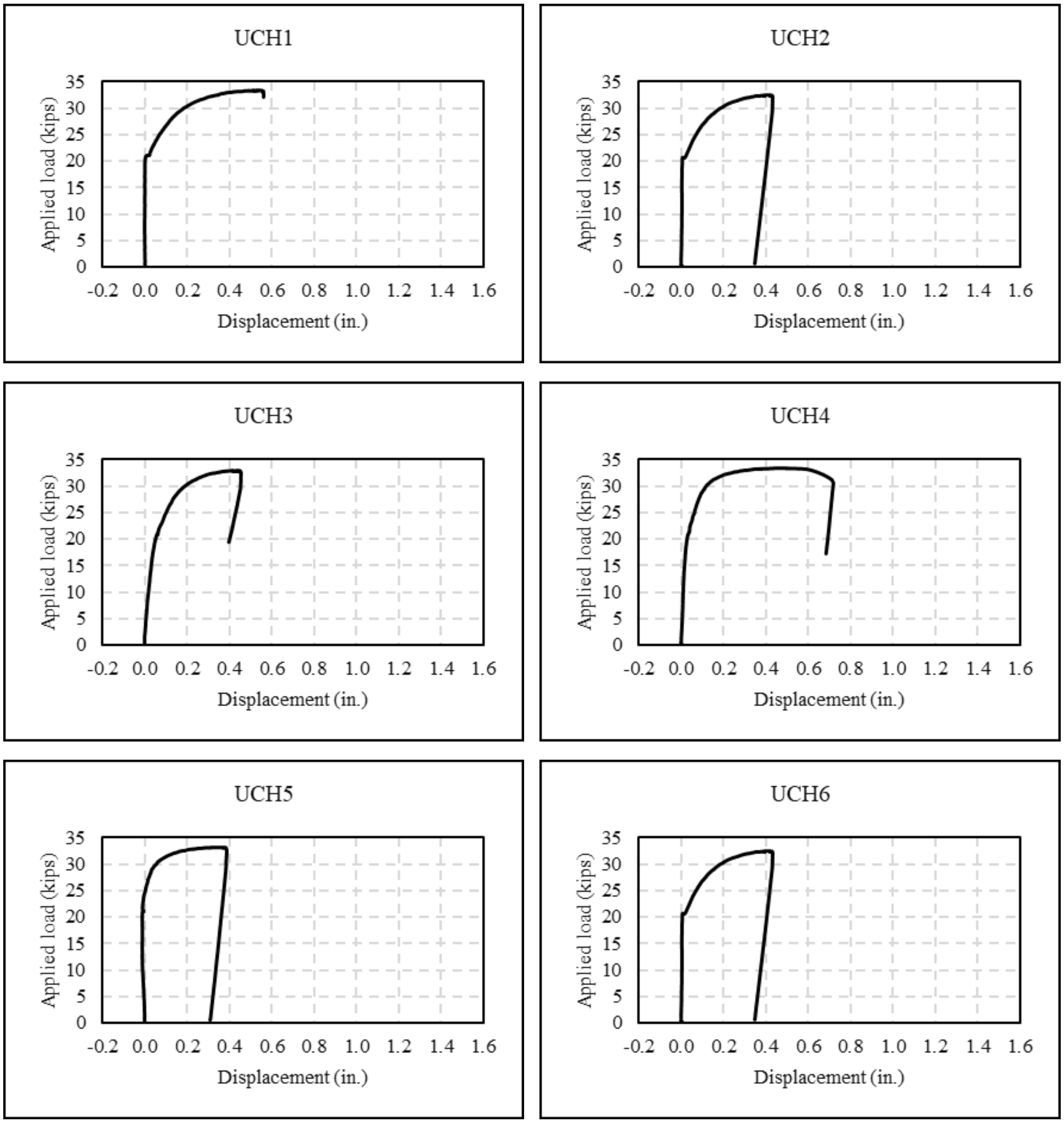


Figure B.6 Applied tensile load vs. displacement plots from test specimens containing uncoated reinforcing bars post-installed with the Hilti HIT-RE 500 V3 chemical adhesive

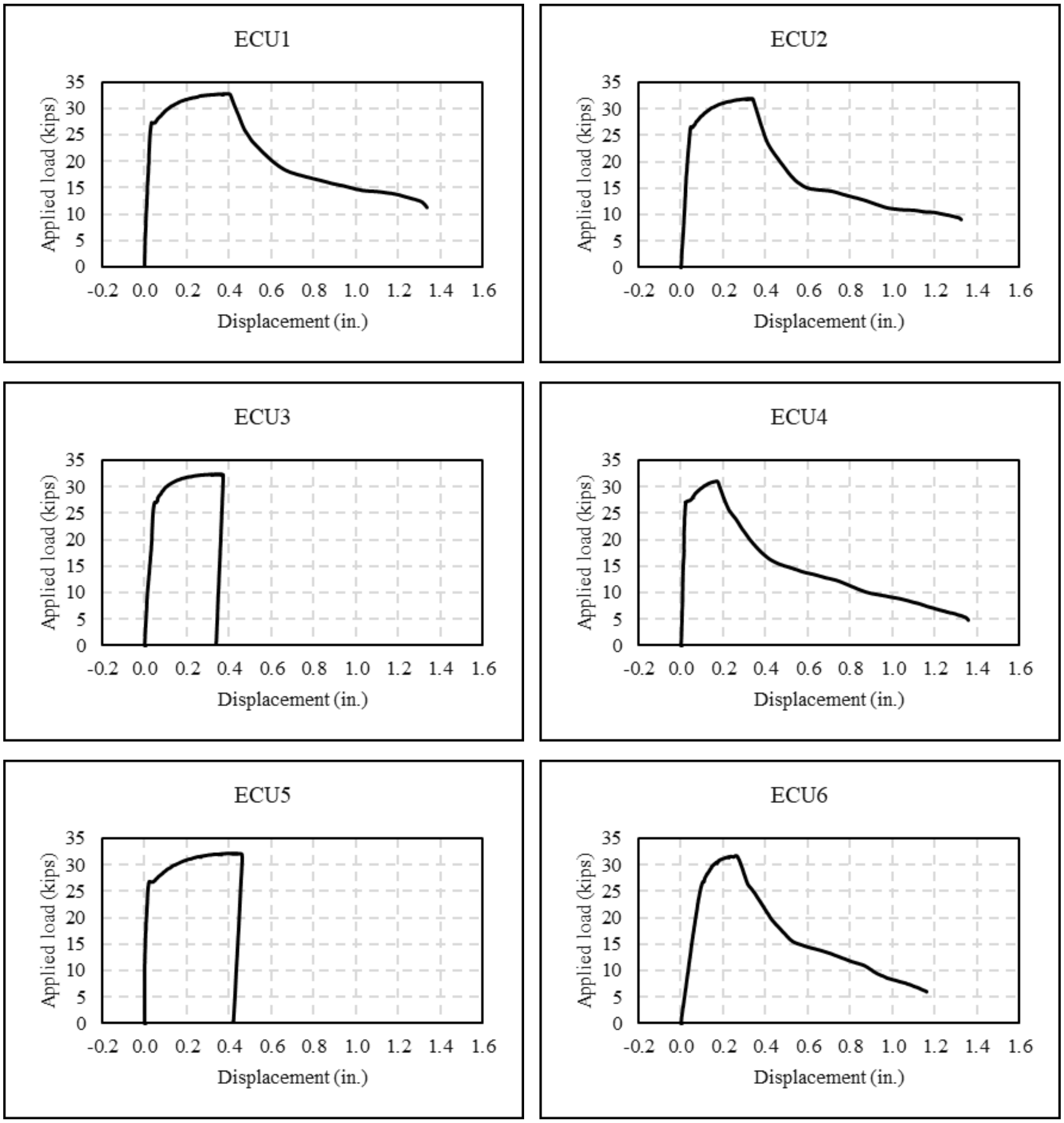


Figure B.7 Applied tensile load vs. displacement plots from test specimens containing epoxy-coated reinforcing bars post-installed with the ATC Ultrabond 365CC chemical adhesive

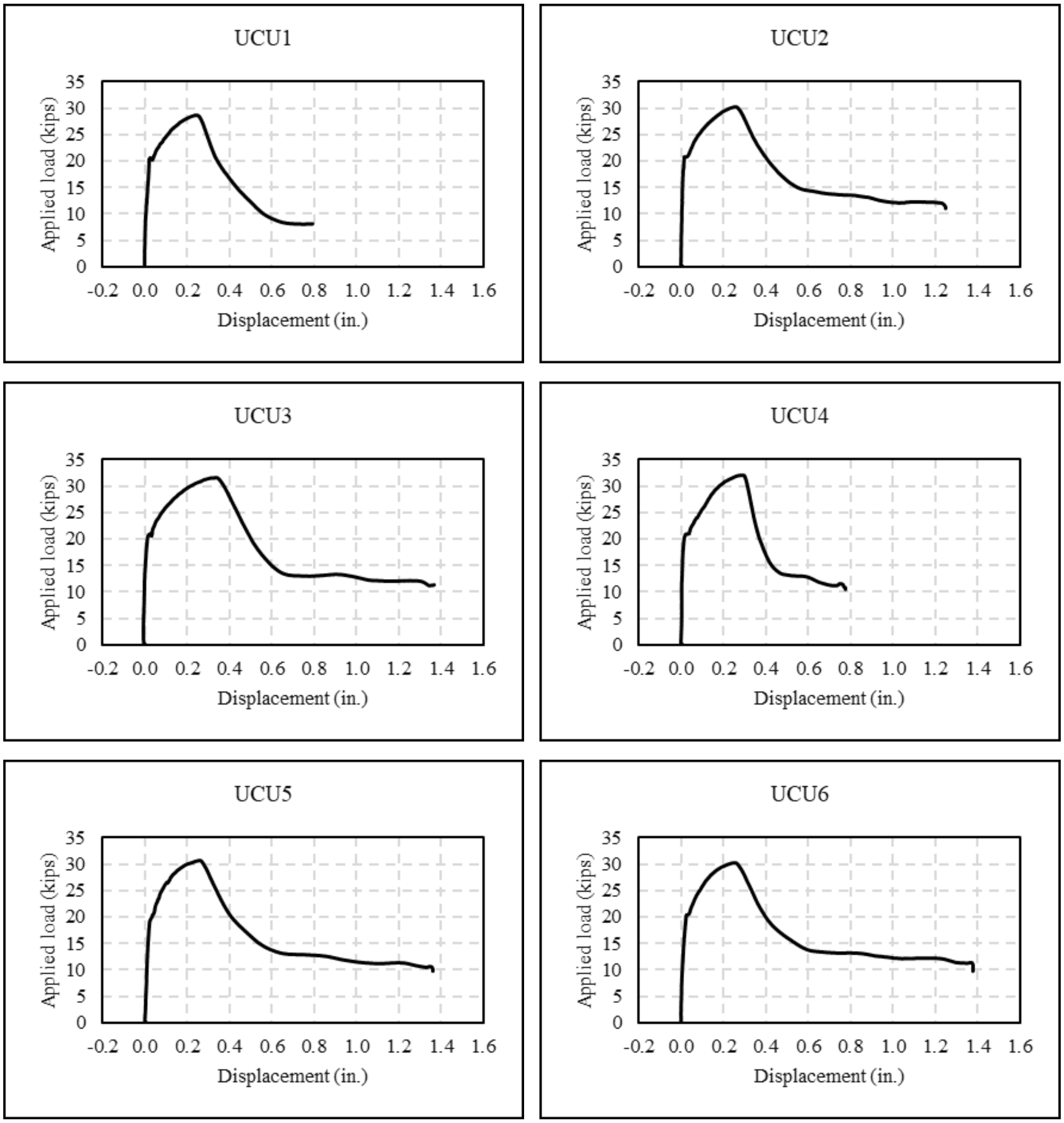


Figure B.8 Applied tensile load vs. displacement plots from test specimens containing uncoated reinforcing bars post-installed with the ATC Ultrabond 365CC chemical adhesive